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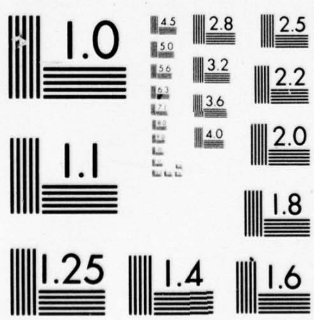

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Stenley E. Jones

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PROCEEDINGS OF A SEMINAR ON URBAN HYDROLOGY Held on

1-3 SEPTEMBER 1970, at
Davis, California,

(11) Sep 70

(12) 294p.

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PROCEEDINGS OF A SEMINAR

ON

URBAN HYDROLOGY

The Hydrologic Engineering Center
Corps of Engineers
Davis, California

1-3 September 1970

FOREWORD

The hydrology of urban areas has become of increasing concern to the Corps of Engineers in studies being made to solve urban water problems. The purpose of this seminar was to discuss current problems in urban hydrology, review methods and techniques being used and outline future research needs. The papers presented represent experience in urban hydrology problems which differ hydrologically, geographically and in extent of urban development.

Papers and discussions are, in general, frank evaluations by the authors and are not official Corps documents. The views and conclusions expressed are those of the individual authors and are not intended to modify or replace official OCE Engineer Regulations, Engineer Manuals or Engineer Technical Letters.

CONTENTS

SEMINAR ON URBAN HYDROLOGY

INTRODUCTORY REMARKS

by

LEO R. BEARD, Director
The Hydrologic Engineering Center

I want to welcome all of you to The Hydrologic Engineering Center and to this seminar on Urban Hydrology. One of the most urgent tasks that face the hydrologic engineer today is the development of techniques and criteria for solving the increasingly complex and pressing problems associated with runoff in urban areas. The continuous development of interspersed impervious areas, of subterranian storm drains, of streets that act as overflow channels, and of major surface drainage channels requires a technology that is quite different from that applicable to natural conditions. While the Corps of Engineers has traditionally been involved only in river and major drainage development, it is becoming increasingly involved in the details of drainage system design.

Problems associated with runoff in urban areas derive not only from the necessity to handle runoff that enters the area from natural watersheds, but also from the necessity to evaluate the effects of urbanization on the actual runoff process. In general, the creation of impervious sections of the drainage area causes an increase in the total volume of runoff and in the peak rates of runoff. Furthermore, the drainage of natural ponding areas, the improvement of natural channels and the realignment of drainage patterns can greatly increase the peak rates of runoff. Adequate technology for evaluating these changes does not exist, and I would therefore like to preface this seminar with a brief and very general discussion of the future technology needs as reflected by present problems.

One of the major problem areas in urban hydrology lies in the overall design of a storm drainage system. Of course, the design cannot be based on historical runoff data, because drainage conditions have been modified from natural conditions and because adequate runoff gaging facilities are never available. It is therefore necessary to compute runoff from rainfall, which can occur in so many significantly different patterns as to almost preclude logical analysis. Perhaps, then, the first need is to develop a technique for analyzing the areal and time patterns of precipitation and to formulate criteria that will reflect rainfall potential as a function of area size and location, timing elements, and recurrence probabilities. This will require elaborate studies of simultaneous rainfall intensities at a number of gage locations.

In order to translate rainfall quantities into runoff, it is necessary to develop a model that will respond to the intricate drainage patterns and to the great variation in loss rates that occur in urban areas. The response sensitivity of such a model should be adequate to reflect the effects of proposed drainage changes, such as storm sewer construction. Recognizing that ponding and surface overflow occur when runoff exceeds sewer capacities, the model must allow for alternate paths of travel from each sewer intake.

Whenever a drainage project increases or redirects runoff quantities, problems of disposing of the runoff can occur. It is not always sufficient merely to dump the runoff into a nearby river. Often that river and rivers downstream can be adversely affected, and remedial measures must be taken. Responsibility for disposing of the water may carry on downstream to the ocean, in many cases.

The types of improvements that must be considered, and hence the hydrologic effects, have increased because of the recent emphasis on esthetic and ecological considerations. Drainage channels through urban areas are often well suited for parks and even water-oriented recreation use. Such use would certainly dictate the hydraulic and hydrologic characteristics of these areas.

A satisfactory hydrologic analysis for a major urban area would involve tremendous amounts of computation and would require large amounts of hydrologic, economic and other data. In order to develop an optimum plan of improvement, it is necessary to consider simultaneously every element from the sewer intakes and laterals to the furthest downstream disposal facility. One can no longer simply design intakes and laterals for maximum prevention of ponding, but must recognize that reducing ponding at the intakes increases flows downstream. Accordingly, a fine economic balance (tempered by esthetic and political considerations) must be obtained in the design.

Runoff from urban areas has increasingly become a quality problem in the receiving rivers. When rainfall washes the cities and surrounding areas, the resulting runoff can contribute to river pollution. Many cities have combined sewers, and overflows often contribute seriously to river pollution. It may be necessary to provide treatment for storm water and the entire structure of the drainage system might desirably be changed.

We recognize that the technology available today is completely inadequate for managing the problems that exist and that will arise in the next few years. We feel that the Corps is becoming increasingly involved in urban drainage problems and that coordination of the overall drainage design might someday become a Federal task. Accordingly,

the purposes of this seminar are to describe the urban hydrology problems that exist, the techniques that are currently employed in their solutions, and the manner in which recent developments in computer technology and numerical techniques might best be applied toward the solutions of the increasingly complex problems. I hope that you will all keep these three objectives in mind as you make your presentations.

We hope that, while you are here, you will become acquainted with the staff at the Center and with the work that we are doing. If there is any way that we can help in regard to your accommodations or travel or other matters while you are here, please let us know.

EFFECT OF URBAN EXPANSION ON HYDROLOGIC INVESTIGATIONS

By

Elliot F. Childs[1]

INTRODUCTION

GENERAL

Numerous cities and towns in New England and probably elsewhere in the United States are frequently experiencing flood problems, not necessarily from rivers, but from small brooks with watersheds varying in size from a few hundred acres to 5 or 10 square miles. Urban expansion in these relatively small drainage areas has, and continues to drastically change the runoff characteristics. Topography that originally was wooded, swampland, meadows or farmland has suddenly been transformed to residential, commercial and industrial developments. The peaceful brook that formerly experienced infrequent floods causing nondamaging inundation of the swamplands and meadows, now is a wall-lined channel or a buried conduit with inadequate capacity for the accelerated runoff.

SCOPE OF PAPER

This paper describes the hydrologic features of a current flood problem being studied for a preliminary reconnaissance report by the New England Division. Particular emphasis is given to the effect of present and future hydrologic changes in the economic evaluation of proposed improvements.

DESCRIPTION

LOCATION

Town Brook in Quincy, Massachusetts with a watershed of about

[1]Chief, Hydrologic Engineering Branch, New England Division

3, 000 acres (4. 7 square miles) will be used in this paper to illustrate the effect of urban expansion on hydrologic and economic investigations. Quincy, a coastal city located about 8 miles southeast of Boston, was first settled in the mid-1600's not long after the Pilgrims landed at Plymouth. The population grew slowly and in 1800 was only 1, 080. By 1900 it had grown to 23, 900 and in the 1965 census the population was over 87, 000.

WATERSHED

Town Brook originates in the neighboring town of Braintree located just west of Quincy (see plate 1). That section of the watershed was rural and parkland until a new superhighway and increasing population demands led to residential developments and shopping centers. Braintree has doubled in population from 16, 400 in 1940 to 34, 000 in 1965. Old Quincy Dam, creating a water supply reservoir for Quincy is located on Town Brook in Braintree. The reservoir with a drainage area of 920 acres has recently been abandoned for domestic water supply purposes, but has some limited industrial use. From the reservoir the brook falls only 50 feet in a distance of 20, 000 feet to the ocean, an average slope of 2. 5 feet per 1, 000.

FLOOD DAMAGE

The drainage system for Town Brook was designed years ago to accommodate the flood runoff from a watershed largely undeveloped and where the headwater topography had considerable effect in reducing peak flows. The expansion in recent years has drastically changed these conditions and the increased runoff is overtaxing the outdated drainage system. Floods will continue to grow in peaks and volume of runoff until the watershed becomes completely urbanized.

There is some cellar flooding and shallow inundation in the residential section of Braintree but the major damages that have been experienced in recent years, lie in a 8, 000-foot section of the brook in Quincy, just over the Braintree line. This section is flatter than average and with an inadequate drainage system of channels and conduits, the excess runoff from the urban growth results in surface storage as shown on plate 1. Floods in March 1968, and again in December 1969 resulted in a pond covering 110 acres and containing about 200 acre-feet of storage. It is reported that the record rainfall accompanying hurricane "Diane" in August 1955 caused even higher stages.

POTENTIAL DAMAGE

Because the channel and drainage restrictions have created this overland flooding in the suburb, the business center of Quincy, located just downstream of the overflow area, has so far escaped major damage. However, the conduit carrying the modified discharge under the center was flowing nearly full in 1968 and overflow in one area threatened a new shopping center. With continued expansion in Quincy and Braintree, a new rapid transit line, currently under construction, and the business center are potentially high damage areas with the ever-increasing floodflows from the upstream areas. It is essential to evaluate this threat in the hydrologic and economic investigation and to determine the feasibility of improvements to alleviate the problem.

HYDROLOGY

RAINFALL

There are no stream gaging stations on Town Brook, nor on any nearby tributaries to determine discharges. Fortunately there is a U.S. Weather Bureau station at the Blue Hills in Milton only 3 miles west of the watershed. The measured hourly values of rainfall and their estimated frequencies at this station for four recent storms affecting flows in Town Brook are shown in table 1. Table 2 shows the comparable hourly values for selected frequencies taken from USWB Technical Paper 40, Rainfall Frequency Atlas of the United States. The estimated frequencies for different durations of the experienced storms shown in table 1 were obtained by relating the measured precipitation with the data from Paper 40.

FLOOD DISCHARGES

To evaluate the effect of the hydrologic changes that have taken place since 1800 on flood runoff, peak discharges were estimated using the rational formula and the rainfall-frequency data from Paper 40. Unit hydrograph procedures were considered but too many assumptions were required to reflect the many variables that define the hydrograph. The rational formula, although of questionable dependability, proved a useful tool for this particular study. Discharges were determined for the total drainage area of 3,000 acres for various degrees of development over the years assuming that

the brook channel had unrestricted capacity.

The rainfall intensity "I" in the formula varied with the time of concentration for different degrees of development over the years. The time of concentration was determined by estimating the velocity of flow in the 20,000 feet of channel from the Braintree area to tidewater. The computed times of concentration reflect the effect of the gradual loss of storage since 1800 and the increase of paved areas and channelization in the watershed. The times of concentration, shown in table 3 vary from 3 hours in 1800 to 1.5 hours in the year 2000. Rainfall values for the fractional hours of concentration were obtained by interpolating the rainfall-frequency data in Paper 40.

The coefficient "C" in the rational formula, Q = CIA, was also assumed to vary: (a) with the development that might have been expected for different dates, and (b) with the rainfall intensity for different frequencies. Table 4 shows the "C" values adopted for the watershed for different dates and rainfall frequencies. It would have been preferable to derive a weighted "C" from dividing the watershed into different types of surface conditions, but there was insufficient information for this refinement.

Thus the only constant in the rational formula was the watershed of 3,000 acres. The other three variables, C, I and Q changed with the rainfall frequency and the selected year of development. The results of combining these factors are shown in table 5 for the year 1970 and on plate 2 for all selected years. It should be noted again that these are theoretical discharges assuming that the channel is unrestricted and would accommodate any magnitude of flow. Obviously this is not true, but the relationship provides useful data for further analysis.

RUNOFF IN EXCESS OF DRAINAGE CAPACITY

The discharge capacity of the present drainage system is 500 cfs, as indicated on plate 2. Flows in excess of this capacity produce overflow, and if unimpeded would flow overland to the ocean. In the case of Town Brook this overland flow is impounded by highway fills and other restrictions and flooding develops as shown on plate 1. As flood losses vary with the depth and extent of this inundation, depth-frequency relationships were determined for different

years and stages of development.

Depth-frequency relationships were computed assuming, (a) all flow in excess of 500 cfs goes into temporary pondage, and (b) the duration of storage accumulation was about 6 hours. This latter assumption was based on observations of the 1969 flood by local residents when maximum depths of flooding were experienced about 6 hours after the period of most intense precipitation. To simplify the mathematics and to obtain a basis for relating depth of ponding to frequency, the volume of storage was determined by taking the natural peak discharges as shown on plate 2, deducting 500 cfs and using the residual as the peak of a triangular hydrograph with a 6-hour base. The area in this triangle represents the volume of storage. Technically the 6-hour base would not remain constant for a wide range of conditions, but it was considered acceptable for this particular study. The amount of water stored for various frequencies in the year 1970 is shown in table 5. Similar analyses were made for other selected years.

The area-capacity curves for the storage area were determined from city maps and depths of water experienced during past floods. The depth of flooding was related to feet, plus or minus the stage of the recent major flood in March 1968 which was used as the reference plane for damage appraisals. The stage corresponding to the required storages for the different frequencies was then determined from the capacity curves and is shown in table 5 for the year 1970. It is interesting to note that zero stage, which corresponds to the level of the March 1968 flood, has an interpolated frequency of about 7 years. This is considered to be a reasonable frequency and a check on the validity of the numerous factors involved in the determination of the precipitation, runoff, and the flood storage.

All the preceding computations to obtain runoff, flood storage, and stage-frequencies were made on the assumption that the drainage facilities, which today limit the total discharge capacity in Town Brook to 500 cfs, were applicable from the year 1800 to 2000. This assumption, obviously not valid, was made in order to illustrate the effect of urban growth only, holding all other conditions constant. The stage-frequency curves were also used to derive the average annual flood losses for Town Brook for the selected dates.

EFFECT OF IMPROVEMENTS

PLAN

Two major improvements are being considered to alleviate flooding on Town Brook, (a) to convert Old Quincy Reservoir, formerly used for water supply, into a flood control and recreation reservoir, and (b) to double the discharge capacity of the main conduit by either a supplemental line through the center of the city, or a diversion conduit that would bypass the center and return to the brook near tidewater. Hydrologic studies were made to determine the effectiveness of the reservoir alone, and then the combination of the reservoir and the increased capacity of the main drainage line.

DESIGN CRITERIA

The Old Quincy Reservoir would be lowered 11 feet during the nonrecreation season to provide 300 acre-feet of storage capacity sufficient to control a 100-year flood. The reservoir level may be raised during the summer months for recreation if this purpose is practical and justified. The flood control outlet will be ungated.

It is not feasible to provide enough conduit capacity to eliminate all damage from shallow inundation in some of the floodprone areas for the 1970 conditions or the year 2000. For the preliminary reconnaissance investigation, it was assumed that the design discharge would be 1,000 cfs, which is double the present capacity of 500 cfs. Steps involved in determining the effect of the improvements for the year 1970 are shown in table 6. The results are shown graphically on plate 5. Effects in other years were derived similarly.

ECONOMICS

The economics of the proposed improvements were being studied concurrent with the preparation of this paper. Although the subject of this seminar and paper is basically hydrologic, it is interesting to note some of the economic results of the hydrologic engineering.

With the reservoir and the increased discharge capacity of the drains, practically all damage would be eliminated in 1970 for a 10-year flood, a substantial reduction for a 25-year flood, and stages experienced in 1968, presently a 7-year event, would become nearly

a 100-year occurrence. In the year 2000, with full development in the basin, damage would be eliminated for a 2-year flood, substantial reductions would be realized for a 5-year flood, and the stages occurring in the 1968 flood would be a 10-year event.

It appears that the design discharge of the new conduit should be further increased to provide a higher degree of protection for the year 2000. Further studies will be necessary to maximize benefits before a final design has been adopted.

The effect of considering future growth on the feasibility of the project is illustrated as follows:

<u>Average Annual Damages (before improvements)</u>

Year 1970	$228,000
Year 1985[a]	$595,000

<u>Average Annual Damages (after improvements)</u>

Year 1970	$18,000
Year 1985[a]	$87,000

<u>Average Annual Benefits</u>

Year 1970	$210,000
Year 1985	$508,000

<u>Average Annual Equivalent Damages for 1970 Plus 15 Years' Growth[b]</u>	$492,000
<u>Average Annual Equivalent Benefits, etc.</u>	$425,000

(a) Economist estimates that nearly complete development will take place by 1985

(b) With discount interest rate at 5.125 percent

Plate 6 shows the increase in annual flood damages from 1800 to 2000, and an enlargement of the years 1970 to 1985 indicating the changes in damages and benefits.

SUMMARY

This paper presents a simple, straightforward procedure used recently at NED for studying the hydrology of a brook undergoing rapid changes in urban expansion. The method described is considered acceptable for reconnaissance reports and possibly with some refinements to meet unusual situations, it may be adequate for authorizing reports. In the example for Town Brook the hydrologic relationships are empirical as there are no stream gaging stations in the watershed. The rational formula is used to compute discharges for different dates of analysis. Urban conditions are projected to the year 2000 in order that the economic justification for improvements will include the increased benefits for the future years as well as the present. This paper has stressed the importance of hydrology in the economic analysis of a project.

TABLE 1

RAINFALL-FREQUENCY-DURATION
STORMS OF RECORD
BLUE HILLS, MASSACHUSETTS

Storm	Duration in Hours 1	2	3	6	12	24	Total
September 1954							
Precipitation (in inches)	0.95	1.83	2.56	4.16	5.03	5.23	5.23
Frequency (years)	1	4	9	30	35	23	
August 1955							
Precipitation	1.74	2.88	4.07	5.27	6.34	9.93	12.77
Frequency	10	35	100+	100+	100+	100+	
March 1968							
Precipitation	0.50	0.99	1.38	2.54	4.30	6.62	7.53
Frequency	1/2	1/2	1	4	15	90	
December 1969							
Precipitation	0.58	1.08	1.48	2.50	3.53	5.56	6.07
Frequency	1/2	3/4	1	3	6	25	

Rainfall frequency interpolated from data in USWB Technical Paper 40 - see table 2

TABLE 2

RAINFALL-FREQUENCY-DURATION
USWB TECHNICAL PAPER 40
QUINCY, MASSACHUSETTS

Frequency in Years	Duration in Hours 1	2	3	6	12	24
1	1.0	1.3	1.4	1.8	2.4	2.7
2	1.2	1.5	1.7	2.3	2.9	3.3
5	1.5	2.0	2.3	2.8	3.4	4.0
10	1.9	2.4	2.7	3.4	4.0	4.7
25	2.1	2.8	3.1	3.9	4.8	5.5
50	2.4	3.1	3.4	4.4	5.2	6.0
100	2.7	3.4	3.7	4.8	5.9	6.8

TABLE 3

TOWN BROOK - QUINCY, MASSACHUSETTS

TIME OF CONCENTRATION

Length of Channel: 20,000 feet, Average Slope: 0.0025

Year	Estimated Average Channel Velocity (ft/sec)	Time in Hours
1800	2+	3
1900	2.5	2.2
1935	2.75	2.0
1970	3.0	1.85
2000	3+	1.5

RAINFALL-DURATION-FREQUENCY

Frequency in Years	Inches of Runoff 1* Hour	1.5** Hours	1.85** Hours	2* Hours	2.2** Hours	3* Hours
1	1.0	1.2	1.25	1.3	1.32	1.4
2	1.2	1.4	1.45	1.5	1.54	1.7
5	1.5	1.8	1.9	2.0	2.06	2.3
10	1.9	2.2	2.3	2.4	2.46	2.7
25	2.1	2.5	2.7	2.8	2.86	3.1
50	2.4	2.8	3.0	3.1	3.16	3.4
100	2.7	3.0	3.3	3.4	3.46	3.7
Applicable Date		2000	1970	1935	1900	1800

* Data from USWB Technical Paper 40
** Interpolated

TABLE 4

TOWN BROOK - QUINCY, MASSACHUSETTS
SELECTED "C" VALUE IN RATIONAL FORMULA
Q = CIA

Frequency in Years	"C" for Different Years 1800	1900	1935	1970	2000
1	0.05	0.10	0.15	0.20	0.25
2	0.08	0.15	0.23	0.30	0.40
5	0.10	0.20	0.30	0.40	0.50
10	0.12	0.22	0.32	0.42	0.52
25	0.15	0.25	0.35	0.45	0.55
50	0.18	0.28	0.38	0.48	0.58
100	0.20	0.30	0.40	0.50	0.60

TABLE 5

TOWN BROOK - QUINCY, MASSACHUSETTS
ANALYSIS OF FLOODS FOR YEAR 1970
BEFORE IMPROVEMENTS

Frequency (years)	Assuming No Discharge Restrictions: Precipitation (inches/hour) (a)	"C" (b)	Flow (cfs) (c)	With Present Discharge Restrictions: Capacity of Drains (cfs)	Excess Flow (cfs)	Excess Volume (ac/ft) (d)	Pondage Stage (ft) (e)
1	0.68	0.20	410	500	0	0	-
2	0.78	0.30	700	500	200	50	-2.3
5	1.03	0.40	1,230	500	730	183	-0.2
10	1.24	0.42	1,560	500	1,060	265	+0.5
25	1.46	0.45	1,970	500	1,470	368	+1.1
50	1.62	0.48	2,330	500	1,830	458	+1.6
100	1.78	0.50	2,670	500	2,170	543	+1.9

(a) From table 3 for 1.85 hours of time of concentration
(b) From table 4 for 1970
(c) Using Rational Formula for area of 3,000 acres
(d) Excess flow divided by 4 for triangular hydrograph with base of 6 hours
(e) From storage curve where "0" stage is level of March 1968 flood

TABLE 6

TOWN BROOK - QUINCY, MASSACHUSETTS
ANALYSIS OF FLOODS FOR YEAR 1970
AFTER IMPROVEMENTS

	Old Quincy Reservoir DA = 920 acres					With Reservoir and Increased Drainage Capacity			
Frequency	Reduced Flows* (cfs)	Capacity of Drains (cfs)	Excess Flow (cfs)	Excess Volume (ac/ft)	Pondage Stage (ft)	Capacity of Drains (cfs)	Excess Flow (cfs)	Excess Volume (ac/ft)	Pondage Stage (ft)
1	300	500	0	0	-	1,000	0	0	
2	505	500	5	1	-5.5	1,000	0	0	
5	875	500	375	94	-1.3	1,000	0	0	
10	1,100	500	600	150	-0.5	1,000	100	25	-3.1
25	1,385	500	885	221	+0.2	1,000	385	96	-1.3
50	1,635	500	1,135	284	+0.8	1,000	635	159	-0.4
100	1,870.	500	1,370	343	+1.1	1,000	870	218	+0.2

* Flows in table 5 reduced in proportion to drainage area controlled plus 20 cfs for reservoir outflow

See other applicable notes in table 5

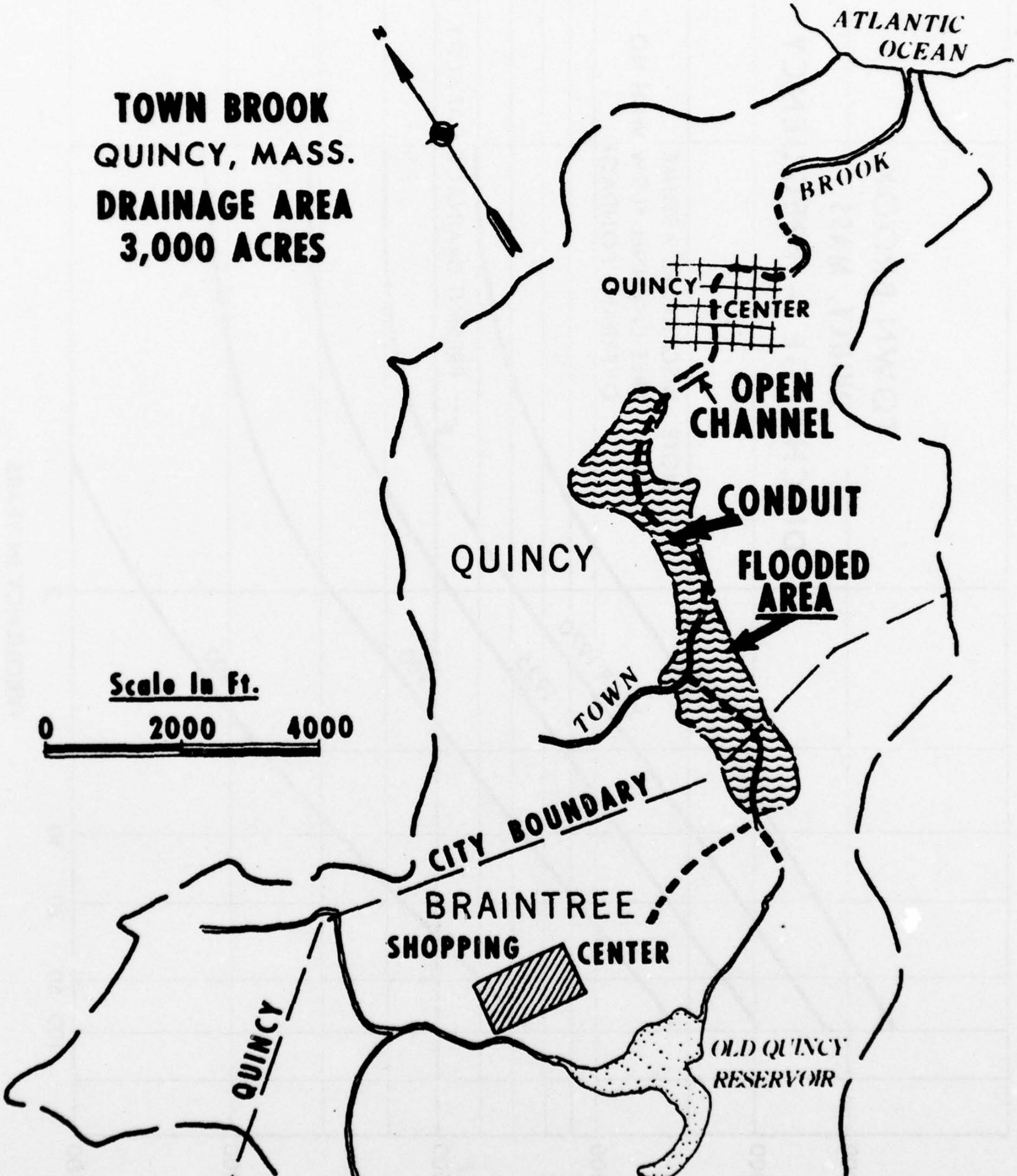
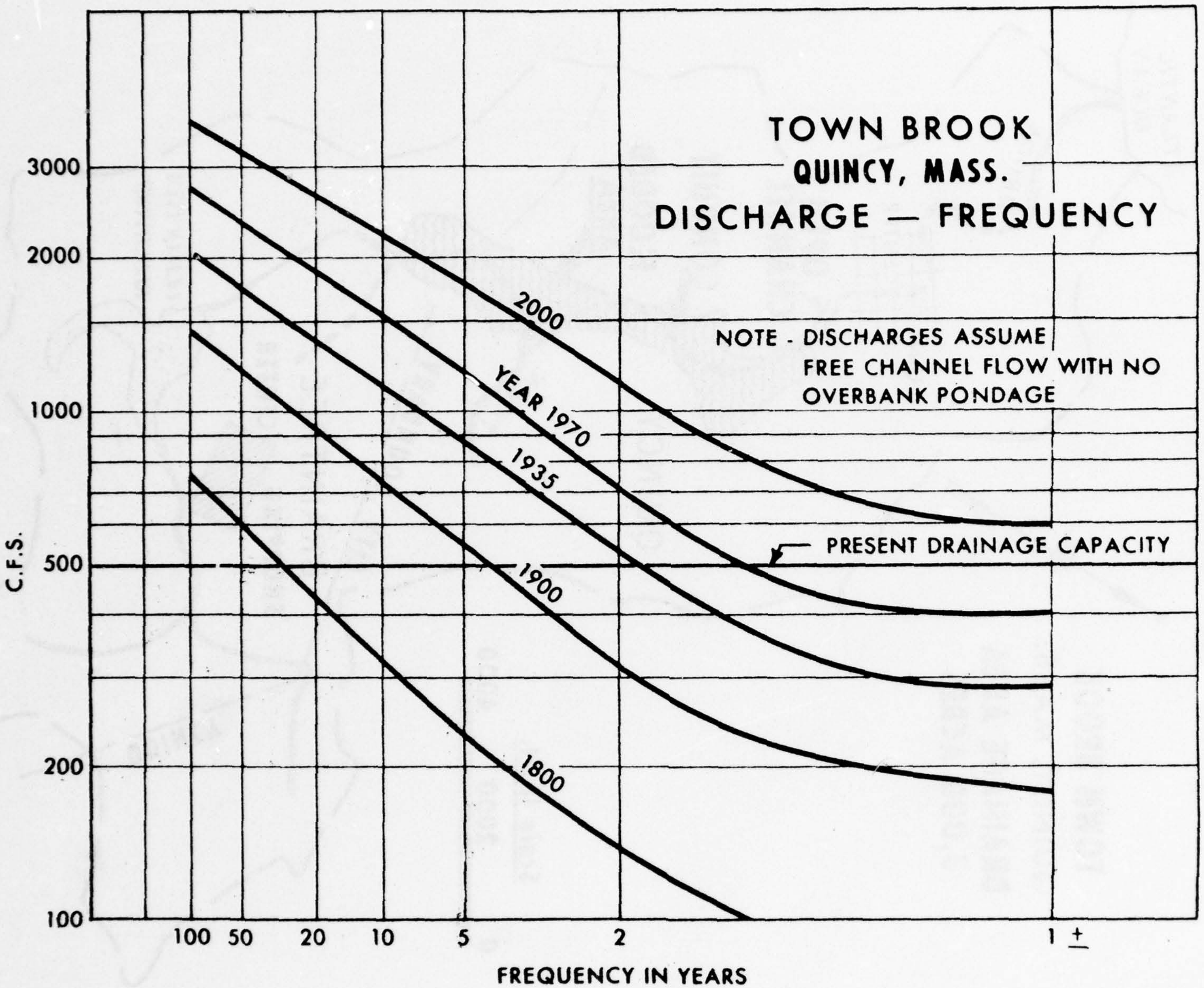


PLATE 1

Paper 1



TOWN BROOK
QUINCY, MASS.
DISCHARGE — FREQUENCY
NOTE - DISCHARGES ASSUME FREE CHANNEL FLOW WITH NO OVERBANK PONDAGE
PRESENT DRAINAGE CAPACITY
2000
YEAR 1970
1935
1900
1800
C.F.S.
3000
2000
1000
500
200
100
100
50
20
10
5
2
1 ±
FREQUENCY IN YEARS

PLATE 2

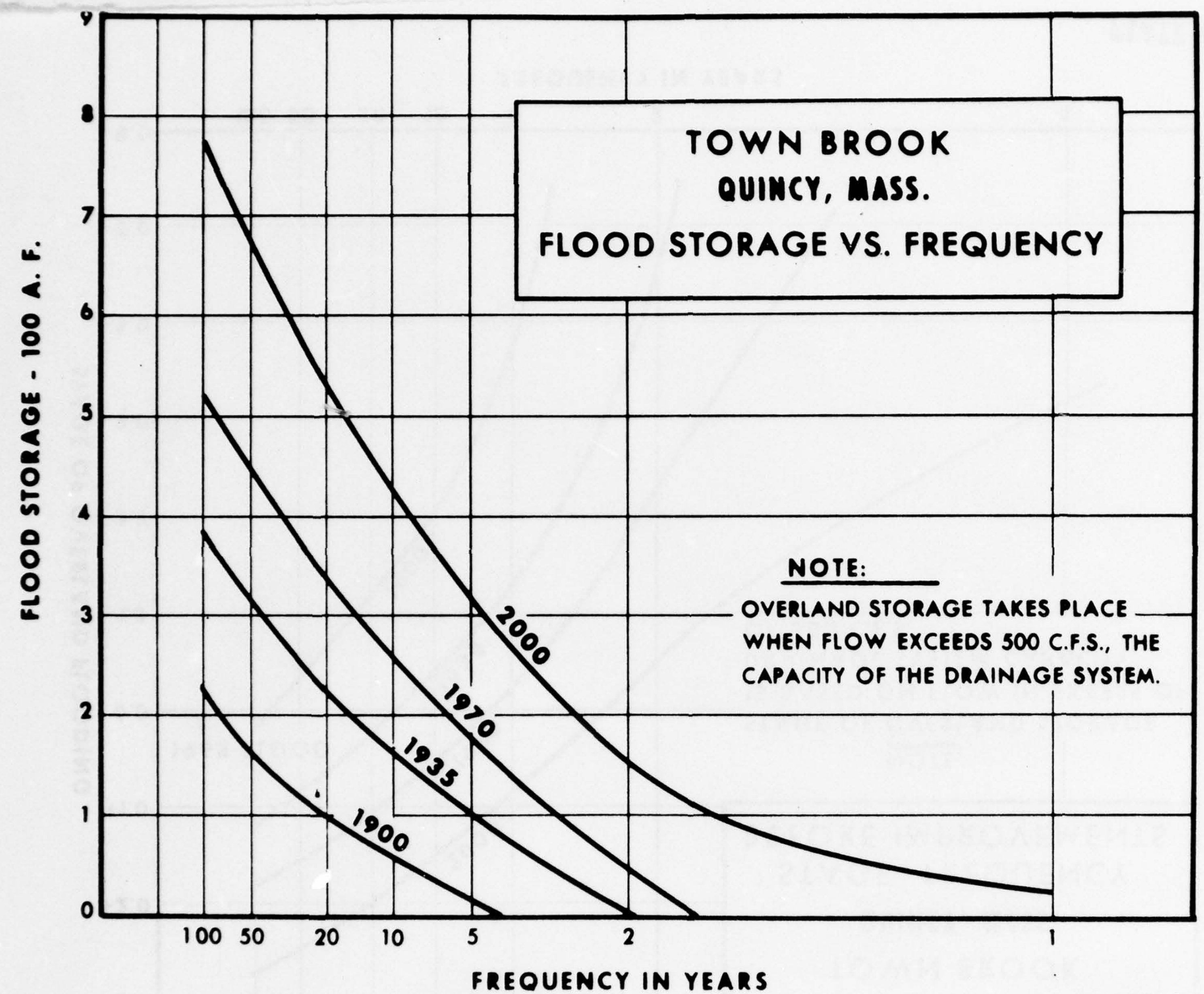

TOWN BROOK
QUINCY, MASS.
FLOOD STORAGE VS. FREQUENCY
NOTE:
OVERLAND STORAGE TAKES PLACE WHEN FLOW EXCEEDS 500 C.F.S., THE CAPACITY OF THE DRAINAGE SYSTEM.
2000
1970
1935
1900
FLOOD STORAGE - 100 A. F.
0
1
2
3
4
5
6
7
8
9
100
50
20
10
5
2
1
FREQUENCY IN YEARS

PLATE 3

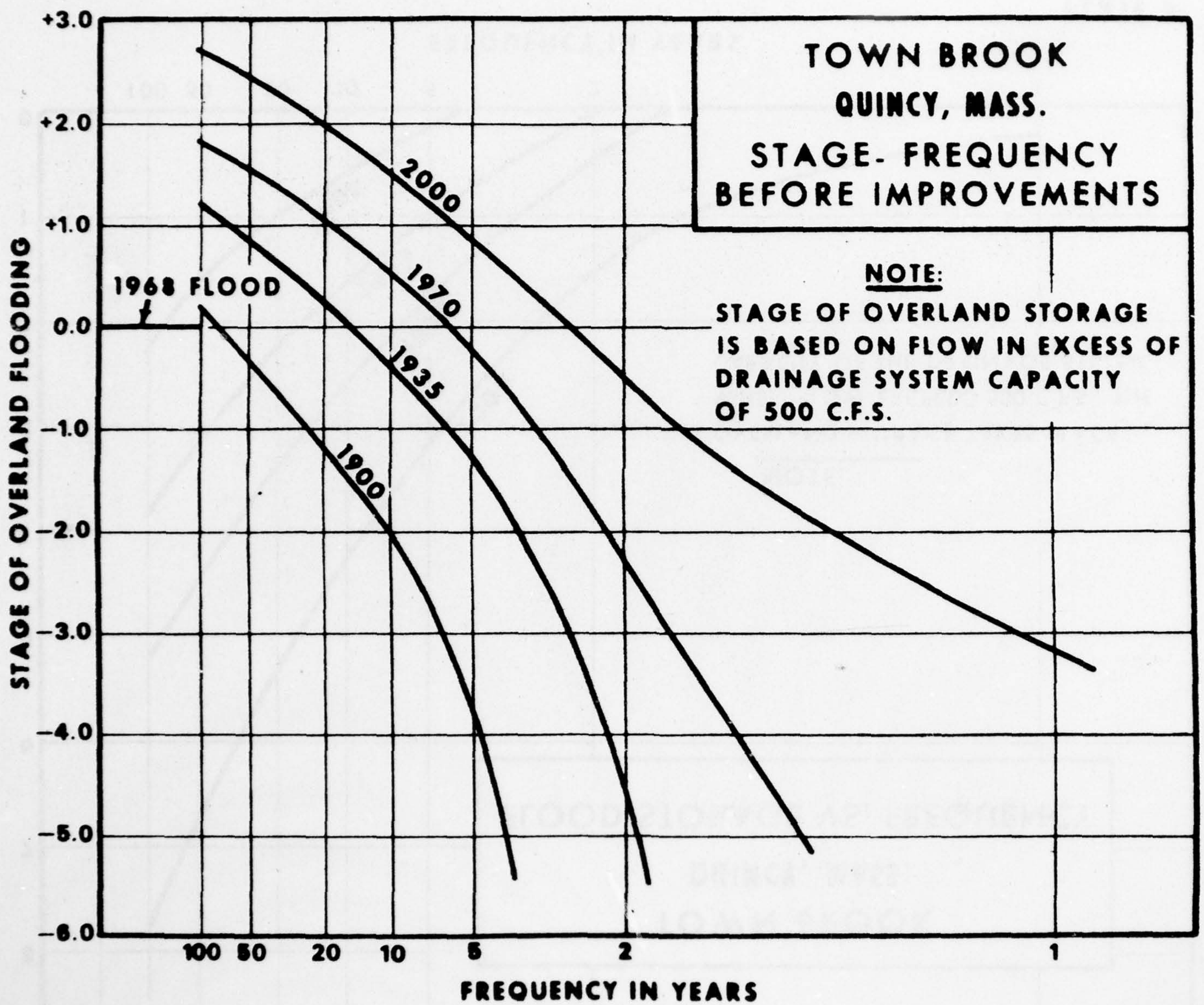
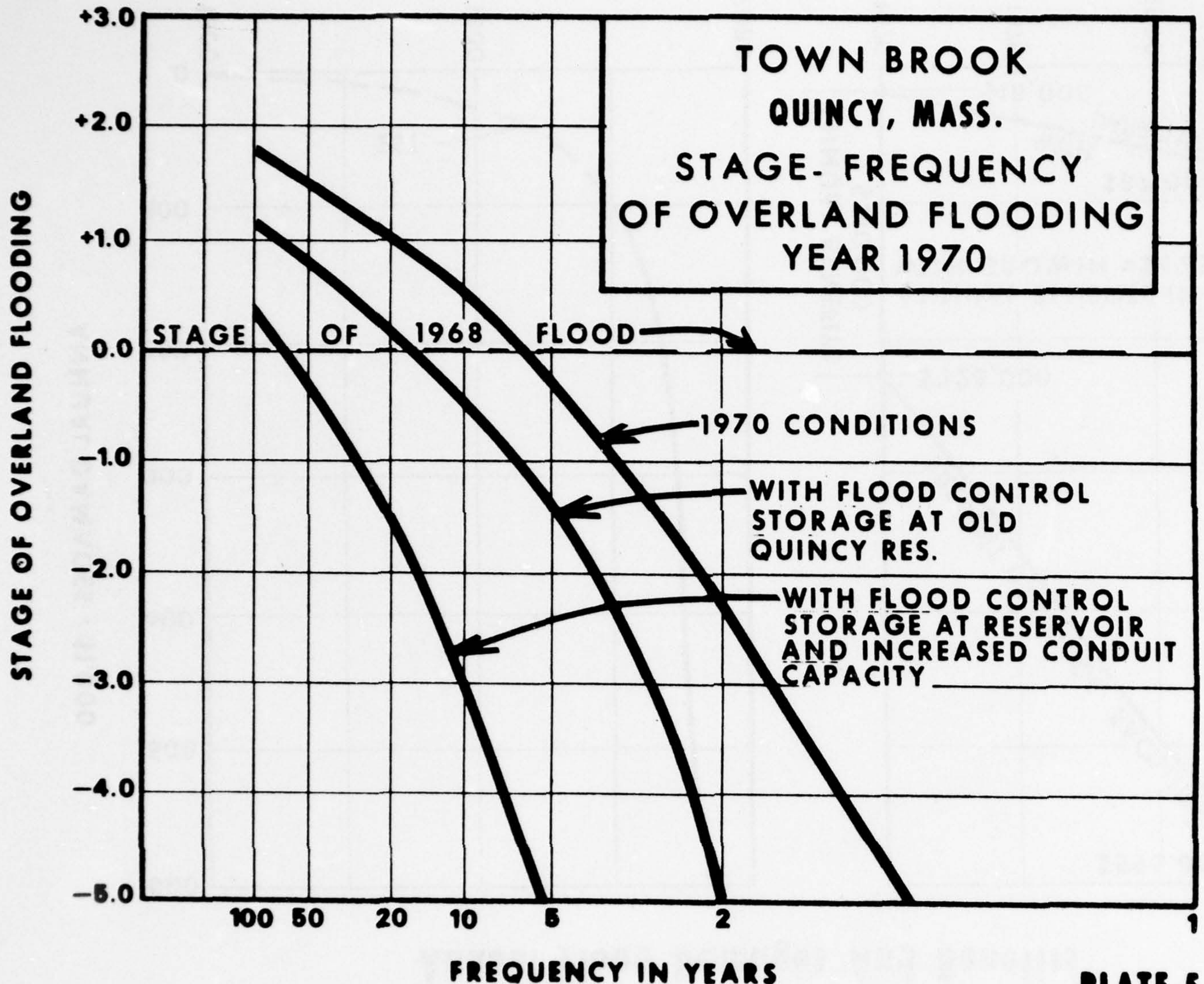

TOWN BROOK
QUINCY, MASS.
STAGE- FREQUENCY
BEFORE IMPROVEMENTS
NOTE:
STAGE OF OVERLAND STORAGE IS BASED ON FLOW IN EXCESS OF DRAINAGE SYSTEM CAPACITY OF 500 C.F.S.
1968 FLOOD
2000
1970
1935
1900
STAGE OF OVERLAND FLOODING
+3.0
+2.0
+1.0
0.0
-1.0
-2.0
-3.0
-4.0
-5.0
-6.0
100
50
20
10
5
2
1
FREQUENCY IN YEARS

PLATE 4



TOWN BROOK
QUINCY, MASS.
STAGE- FREQUENCY
OF OVERLAND FLOODING
YEAR 1970
STAGE OF 1968 FLOOD
1970 CONDITIONS
WITH FLOOD CONTROL STORAGE AT OLD QUINCY RES.
WITH FLOOD CONTROL STORAGE AT RESERVOIR AND INCREASED CONDUIT CAPACITY
STAGE OF OVERLAND FLOODING
+3.0
+2.0
+1.0
0.0
−1.0
−2.0
−3.0
−4.0
−5.0
100
50
20
10
5
2
1
FREQUENCY IN YEARS

PLATE 5

TOWN BROOK - QUINCY, MASS.
Annual Flood Damages And Benefits

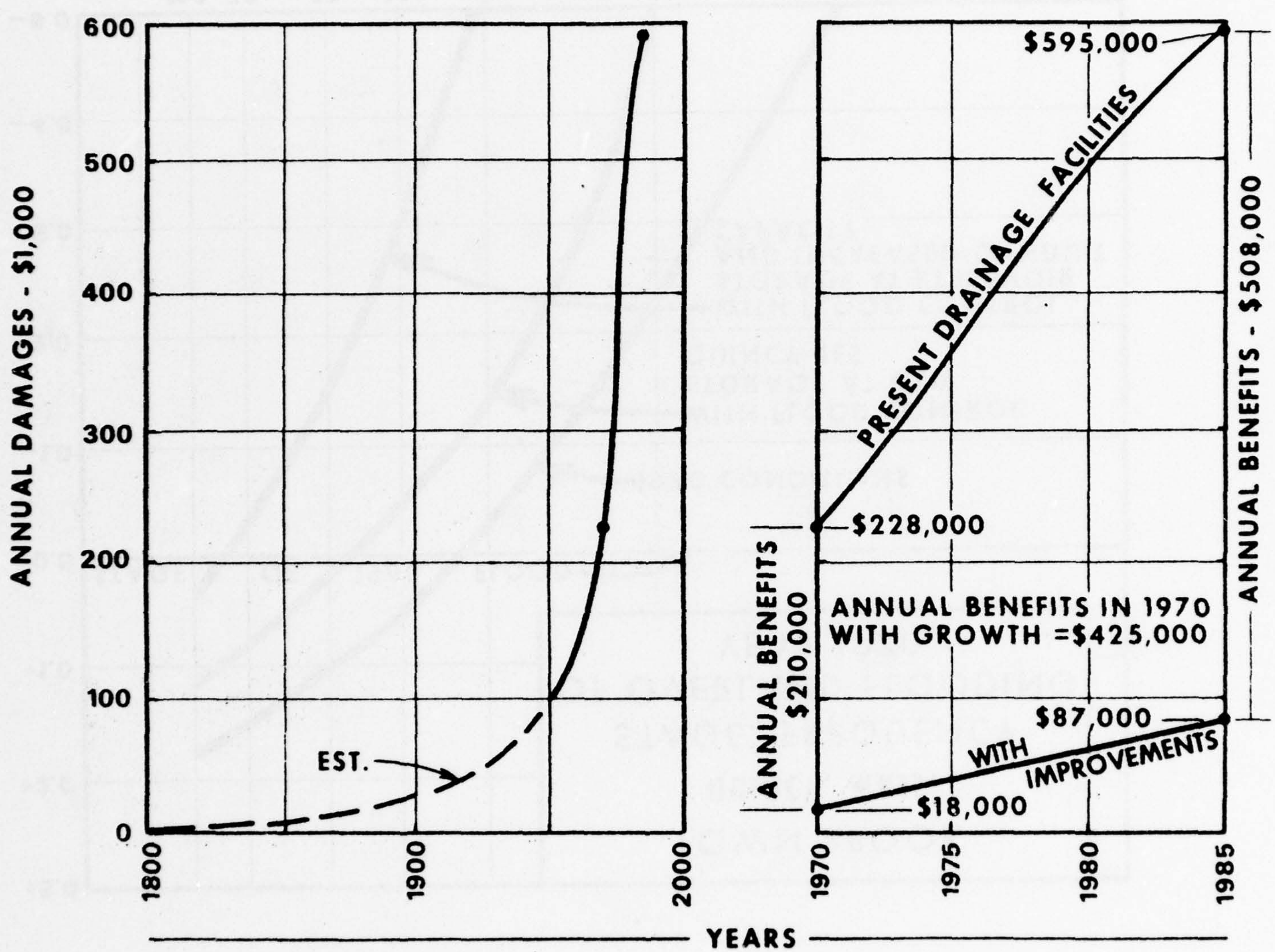


PLATE 6
Paper 1

EFFECTS OF URBAN EXPANSION
ON HYDROLOGIC INVESTIGATIONS

Discussion

Question, Mr. Holler: How were runoff coefficients determined?

Reply, Mr. Childs: Empirically. The use of the Rational Formula and the assumed coefficients was to illustrate the relative effect of the urbanization on runoff. Peak discharges were not used for design nor for economic analysis. Volume of runoff, as affecting storage and depth of flooding, was equally important in this preliminary planning.

Question, Mr. K. Johnson: Was any consideration given to varying T_c with frequency?

Reply, Mr. Childs: No, but no doubt the time of concentration would vary with the amount of rainfall and intensity. Varying the "C" value with frequency probably already compensates in part for this factor.

Question, Mr. K. Johnson: What about the possibility of a check on standard deviation to assess variability of C?

Reply, Mr. Childs: We have no discharge records in the area to measure flow from an urban area, hence have no data for determining a standard deviation.

Comment, Mr. Northrop: A description of the soil characteristics related to infiltration capacity would add to the paper from the national viewpoint. Mr. Childs indicated the basin has a large capacity for infiltration unless saturated by antecedent rainfall. It is important to recognize the high variability in infiltration capacity both locally and regionally. For example, clay soils as hardpan in Missouri have a low infiltration capacity, while Mr. Johnson pointed out some sandy soils in his area (Nebraska) had considerable retention capacity.

Comment, Mr. Northrop: In my opinion, the engineer has a responsibility to inform the city of the magnitude and probability of the Standard Project Flood (SPF) and recommend that overland flowways be provided, even if not economically feasible to contain it without damage.

Reply, Mr. Childs: We will check to determine the effect of the SPF. It may not be as bad as first thought. However, we will have to use considerable tact and discretion before alarming the city officials with the estimated effect of a large synthetic flood.

Question, Mr. Beard: You pointed out the high cost of increasing conduit size because of utility relocations. Has any thought been given to a deeper tunnel that would take only the overflow from the existing conduit?

Reply, Mr. Childs: Yes, we have discussed this possibility, but costs probably would be prohibitive. However, we will explore this alternative. Possibly, one tunnel might intercept the flow on three brooks which would make it more feasible.

Question, Mr. W. Johnson: Is the financial feasibility for local cost sharing being considered in developing alternative means to solve the flooding problem?

Reply, Mr. Childs: I presume that the Project Engineer in the Small Project Section will consider alternates in presenting protection plans to the city of Quincy. There are no low cost alternatives to provide adequate protection, but I am sure various costs, and degree of protection provided, will be furnished the city. Their acceptance of a plan will no doubt depend considerably on the distribution of costs between Federal and local interests.

EFFECTS OF URBAN DEVELOPMENT ON STORM RUNOFF RATES

by George S. Hare[1]

Acknowledgment

Much of the historical information included in this paper was developed in collaboration with Donald VanSickle, Vice-President of Turner, Collie and Braden, Inc., Consulting Engineers, of Houston, Texas. Mr. VanSickle presented a paper on urban hydrology at the training course in Flood Plain Management at the Hydrologic Engineering Center in Sacramento, California in March 1968.

Mr. VanSickle has been quite active in the field of urban hydrology in recent years and has conducted a number of hydrologic studies of urban areas and the interior drainage design analyses for the Corps of Engineers Hurricane Flood Protection Project at Port Arthur, Texas.

His contribution to the field of urban hydrology is noteworthy and we welcome the opportunity during this seminar to consider some of his concepts and approaches to the problem.

Introduction

A substantial part of the efforts of the U. S. Army Corps of Engineers in its water resources planning and construction activities has been directed toward urban areas. Many harbor and navigation facility improvements have been provided for the various urban areas around the country. Similarly, many of the Corps multi-purpose reservoirs provide water supply storage, flood control, hydroelectric power, and recreation for nearby urban areas. Many urban areas are protected by these reservoirs and other flood control facilities

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from the ravaging effects of river floods resulting from heavy runoff from upstream watersheds. Indeed, a large part of the benefits from such flood control are derived from the prevention of flood damages to the urban areas. Other protection projects benefitting the urban areas include hurricane flood protection systems for various cities on the Gulf and Atlantic coast and the various river control and levee projects along the Mississippi, Missouri, and other interior rivers.

In a great many cases, however, the design of these facilities, while directed toward protection of the urban areas, is actually a function of runoff from rural and undeveloped areas in the upper parts of the watersheds. In relatively few instances is the effect of runoff from the urban areas themselves a major factor in the design of the improvement facilities. As a result, the vast majority of the effort in hydrologic studies for such planning activities has been directed towards the determination of storm runoff rates from undeveloped areas and the routing of these floods through the various reservoirs or channel sections to be improved.

The rapid migration from rural to urban areas in recent years and the overall population increases have greatly expanded the areal extent of urban developments. Further, the trend toward suburban living has extended the total urban area of most cities for many miles in all directions. Under such circumstances the design of water supply and flood control facilities must take into account the existing and projected future effects of this urban development on storm runoff rates. The purpose of this discussion is

to recognize and describe these effects. Some of the developments in urban hydrology over the past 60 or 70 years will be outlined briefly, and then some recent developments in the study of urban hydrology will be described in more detail, insofar as they may be applied to the design of such facilities as hurricane protection projects, flood control channels, stream rectification works and flood plain management or control activities in urban areas.

General Effects of Urban Development on Storm Runoff

Urban development of the watershed basically affects drainage characteristics in two ways: (1) reduction in infiltration losses which results from covering the permeable soils with impermeable streets, parking areas, roofs, etc., and (2) provision of hydraulically more efficient channels through which the storm runoff can flow. These factors result in an overall increase in storm runoff because of the reduced infiltration losses and higher peak runoff rates because of shorter concentration time in the more efficient drainage systems. While the increase in total runoff is of significance in some areas, particularly in those areas where sandy or otherwise very permeable soils occur, generally the most significant effect of urban development is the sharp increase in peak storm runoff rate, resulting from the reduced concentration times. This effect is illustrated in figure 1. While it is not difficult to determine that urban development generally increases both the total runoff and the peak runoff rates, it has been extremely difficult to develop relationships which accurately define the extent of these changes.

Although research on the subject has been performed spasmodically over at least 80 years, much of the intensive effort has been concentrated within the last 15 years. The following section provides a brief outline as prepared by VanSickle of the historical developments in urban runoff design.

History of Urban Hydrology

In 1889 Kuichling published in the Transactions of the American Society of Civil Engineers, an article entitled "The Relation Between the Rainfall and the Discharge of Sewers in Populous Districts," in which he presented a method for determining the storm runoff rate based on rainfall. This method has, over the years, been given the designation of the "rational method." While some improvement to the method has been accomplished, the procedure is still basically the same as presented by Kuichling. In fact, many millions of dollars worth of storm sewers are designed each year throughout the United States on the basis of this rational method relationship. The rational method procedure was adopted almost universally for storm sewer design, and no further advances in technique were undertaken until the 1930's when the work of Sherman on the unit hydrograph led to significant developments in the science of hydrology.

In 1932 Sherman formulated the unit hydrograph theory which, with some modification, is now the almost universally-accepted method of determining the runoff hydrograph from the effective rainfall, at least in the field of rural hydrology. The unit hydrograph theory assumes that the runoff hydrograph, due to one inch of effective rainfall, generated uniformly in space

and time over the watershed in a unit period, is a characteristic of the watershed itself.

Three studies in the late 30's and early 40's attempted to go beyond the simple rational method procedure for computation of urban runoff by very detailed analyses of recorded rainfall-runoff events in several urban areas. Horner and Flynt in Transactions of the American Society of Civil Engineers, 1936, in an article entitled "Relation Between Rainfall and Runoff from Small Urban Areas" made some attempts to utilize unit hydrograph theory in the analysis of runoff data for the St. Louis area. Horner and Jens in an article entitled "Surface Runoff Determination from Rainfall without Using Coefficients" in the 1942 Transactions of the American Society of Civil Engineers, reported on further development of the design procedures and further experimental data. However, the latter paper made no further use of the unit hydrograph approach, but concerned itself more with determination of overland flow rates, losses due to infiltration and modification of the flow hydrograph through gutter storage and conduit storage. Hicks, in an article entitled "A Method of Computing Urban Runoff" which appeared in the 1944 Transactions of the American Society of Civil Engineers pursued a very similar approach to that used by Horner and Jens in 1942. Here again, the concentration appeared to be on gutter detention, conduit detention, surface losses, infiltration losses, etc., but no significant effort was made to utilize unit hydrograph theory in the analysis of the data or in the development of proposed design procedures. Because of the very complex nature of the design procedures proposed by Horner, Flynt, Jens, and Hicks, their

design procedures do not appear to have been accepted to a significant degree by design engineers over the intervening 25 years. Urban drainage systems continue to be designed utilizing the rational method.

During the 1940's and early 1950's substantial contributions to hydrologic design practices using the unit hydrograph theory were made by the Corps of Engineers and other water resource development agencies. Snyder's synthetic unit hydrograph method was developed during this period and found substantial application to the design of water resource projects in this country and elsewhere in the world. In the 1960 Transactions of the American Society of Civil Engineers, Tholin and Keifer reported on "Hydrology of Urban Runoff." This paper covered the development of the "Chicago Hydrograph Method" for the design of storm sewer systems, and enabled engineers to apply modern design procedures to the complex problem of storm sewer design.

Another major contribution to the unit hydrograph approach for the design of urban drainage systems was presented by Eagleson in the Journal of the Hydraulics Division of the ASCE in March 1962 in a paper entitled "Unit Hydrograph Characteristics for Sewered Areas." In this paper Eagleson analyzed the data obtained by the Corps of Engineers in Louisville, Kentucky, in the late 1940's. The data had been obtained to develop design hydrographs for specific drainage pumping plants in the Louisville area, but the results of these studies had not been published and were generally not known to the rest of the profession. Eagleson's analysis related the peak discharge and lag time of the unit hydrograph to watershed characteristics which were defined by a parameter.

In the early 1960's the U. S. Geological Survey, in cooperation with a number of local metropolitan area agencies, began a program of stream gaging in small urbanized areas, together with appropriate control gaging networks, with the purpose of obtaining much needed data on the urban runoff phenomenon. Projects have been under way for sometime now in Philadelphia, Pa., Austin, Texas, Houston, Texas, Dallas, Texas, Alexandria, Va., and Nashville, Tenn. The data from these projects have been published in the form of open file reports. The information being gathered by the City of Houston and the U.S.G.S. will be described later.

Urban Runoff Design

It is not the intent of this discussion to go into details as to the methodology applicable to development of urban runoff design criteria. However, the results of VanSickle's studies and those conducted by the Galveston District in the Houston area will be reviewed.

The Corps of Engineers has used the basic unit hydrograph method extensively in development of hydrology for its civil works projects. Results obtained by this method are reliable and acceptable when proper coefficients are used. Although considerable data has been accumulated on which to base values of C_t and $640C_p$, practically all of these data reflect runoff characteristics of relatively undeveloped rural areas. During the course of studies of the major drainage channels in the Houston area, VanSickle analyzed recorded storms from a number of watersheds which varied from undeveloped to almost fully developed in character. He found that Snyder's unit hydrograph coefficients changed considerably with the stage of urban development even for areas with similar topographic features.

Hydrologic studies for Brays Bayou in Houston were begun as early as 1939, when little of the watershed had been subjected to urban development. Consequently, it was possible to analyze the effects of urban development on a single watershed over a period of about 30 years, from its very earliest stages of development through almost complete devotion of the watershed to urban interests.

As a result of his Brays Bayou studies, VanSickle noted that the peak discharge for a given watershed, or for two watersheds with comparable L and L_{ca} values, is a function of the ratio of the coefficients $640C_p$ and C_t. He also noted that the change in ratio of the coefficients more accurately describes the effects of urban development than the variations in the coefficients themselves. Figure 2 indicates the changes in unit hydrograph characteristics for the Brays Bayou watershed.

As an example, VanSickle found the coefficients for the Brays Bayou watershed in 1939 to be 211 and 2.7 for $640C_p$ and C_t, respectively, resulting in a ratio of 78. In 1950, during design studies for the Brays Bayou channel improvement, the coefficients were determined to be 180 and 1.8 for a ratio of 100. By the June 1960 storm, the coefficients indicated a change to about 70 and 0.3 for a ratio of about 240. During this time, the unit hydrograph peak increased from some 1800 c.f.s. to about 4500 c.f.s. and the time to peak had decreased from 12 hours to 3 hours.

Even though these results are by no means conclusive, they are indicative of the drastic changes in runoff production brought about by complete urbanization of the watersheds.

Similar type studies conducted by the Galveston District in 1963 for White Oak and Buffalo Bayous in Houston produced somewhat similar results.

A detailed study of ten storms occurring during the period 1952 to 1961 indicated some general trend toward increased discharge rates; however, increases were not drastically different as noted by VanSickle for Brays Bayou.

The present urban developments in the White Oak Bayou and Buffalo Bayou watersheds are similar to those in the Brays Bayou area; however, adequate records are not available for detailed study of the watersheds under the early stages of urban development as they were for Brays Bayou.

The results of the White Oak Bayou studies are shown on figure 3. We notice that the values of $640C_p$ and C_t ranged from 168 and 1.17, respectively, in 1952 to 235 and 1.41 in 1960. Peak unit hydrograph discharge rates varied from 1893 c.f.s. to 2407 c.f.s.

On Buffalo Bayou, during the period 1956 to 1961, we found no significant change in coefficient values except for random variation due to the inaccuracies in rainfall determinations and storm reproduction procedures. Results of this study are shown on figure 4. Buffalo, White Oak and Brays Bayou watersheds are outlined on figure 5.

U.S.G.S. Gaging Network, Houston

A relatively good network of rainfall and streamflow gages has been established throughout the Houston area. In 1964, the City of Houston and U. S. Geological Survey entered into a cooperative agreement to gather data on small urban drainage areas varying in size, shape, topography and degree of urbanization. The area of study comprises an approximately 30-mile square area of gently sloping, almost featureless plain. Land surface elevations range from about 35 feet above mean sea level in the southeast to 135 feet in the northwest. The 30-year average rainfall for Houston is 45.95 inches,

with a fairly uniform distribution throughout the year. Annual rainfall varied from a maximum of 72.86 inches in 1900 to a minimum of 17.66 inches in 1917.

The system of gages is illustrated on figure 6. The study will involve the collection of runoff data at 33 sites. Gaging stations for rainfall and runoff data were established in 1964 at 23 sites having drainage areas ranging in size from 0.1 to 35.5 square miles. Ten other stations to collect streamflow data from areas of 24.7 to 359 square miles were established prior to 1964.

Results of the initial studies that we have conducted in the Galveston District using these data are not conclusive; however, there is an indication, as VanSickle noted, that values of unit hydrograph coefficients vary considerably between watersheds and with the degree of urbanization and, in many cases, are substantially different from values that we consider appropriate at the present time.

Houston Design Criteria

VanSickle, after analysis of one year of records on the Houston gaging network, concluded that no consistent pattern of synthetic unit hydrograph coefficients could be developed which would adequately relate the unit hydrograph properties to the degree of urbanization of the watershed. Subsequently, he suggested that a basin factor, K, could be described which would include most of the significant drainage basin characteristics which affect the runoff process, while at the same time providing the best correlation with the key hydrograph parameters of peak discharge and time to peak.

While the methods and procedures for analyzing urban runoff as described by VanSickle do not necessarily represent the present thinking of the Corps

of Engineers, it behooves us, as hydrologists, to consider new methods and their applicability to the urban runoff problem. It is believed that this approach is applicable in similar type urban areas over the country. Mr. VanSickle's original description of the "basin factor" method of analysis is presented in the following paragraphs for your consideration and comment relative to its application to the Corps of Engineers hydrologic investigation program for urban areas. The basin factor, K, is defined as follows:

$$K = \frac{(L_t)\ (\overline{L})}{\overline{S}^{\frac{1}{2}}}$$

where: L_t is the total length of drainage channel in the basin, in miles, and includes all storm sewers 36 inches or larger and all drainage channels large enough to be shown on U. S. Geological Survey contour maps. L_t divided by the drainage area is defined as the drainage density. $\overline{L}$ is the mean basin length, in miles, and $\overline{S}$ is the mean basin slope. These latter two quantities are determined as follows utilizing topographic maps of the area:

1. Prepare a curve showing the relationship between length along the main channel and the cumulative drainage area.
2. Prepare a curve showing the relationship between natural ground elevation and cumulative drainage area for the watershed. Both of these curves should represent accumulation of area in an upstream direction, similar to those in figure 7.
3. The area under the length-area curve divided by total area gives $\overline{L}$.
4. The area under the elevation-area curve divided by area gives the mean basin rise $\overline{H}$.
5. $\overline{H}$ in feet divided by $\overline{L}$ in feet gives mean basin slope $\overline{S}$.

Mean basin length and mean basin slope are functions of topography and are essentially unaffected by urbanization. Total channel length increases with urbanization, because of construction of storm sewers and drainage ditches, although rectification works may actually reduce main channel lengths.

Figure 8 shows a typical area in the gaging program, with the drainage area and drainage density. Figure 9 shows a typical storm hydrograph and rainfall accumulation for the fully urbanized Willow Waterhole Bayou watershed. Note the very sharp peak and sudden decrease following the rainfall because of the efficient storm sewer system.

Data from the gaging stations for which satisfactory storm records were available were computed and are tabulated in figure 10. Each of the basins is given a classification A, B, C, or D which generally defines the degree of development as follows:

A. Fully-urbanized - well developed drainage system including storm sewers.

B. Fully urbanized-poorly-developed drainage system - mostly open channels.

C. Partially urbanized - few drainage facilities of any type.

D. Undeveloped with only natural drainage system.

The basin factors and time to peak were plotted, indicating the relationship shown in figure 11, while figure 12 shows the relationship between basin factor and peak discharge. While these relationships are based on a rather limited amount of data from only the Houston area, it is considered that they may be applicable in other parts of the country because of the character of the basin factor. Studies are under way incorporating additional data from Houston as well as from other areas in an attempt to establish greater confidence in their overall applicability. Certainly, they indicate the significance of effect of urban development on storm runoff.

In order to apply these data to design, the following procedures are utilized:

1. Develop the mean basin length, $\bar{L}$, and mean basin slope values, $\bar{S}$, from topographic maps.
2. Estimate the degree of development anticipated in the watershed for the period of design. Predictions of land use may indicate complete urban development or, because of certain limiting conditions such as park areas, oil fields, land unsuitable for development, etc., only partial development.
3. Estimate drainage density for each of the areas of use. In the Houston area undeveloped rural areas have drainage densities on the order of 0.8 to 1.0 miles per square mile, while for fully developed storm sewered areas the density is approximately 5.0 miles per square mile. Intermediate degrees of development vary almost linearly between these limits.
4. Multiply the drainage density by the area for each type of area to get total channel length, and add these to get total channel length for the basin, L_t.
5. Determine the basin factor, K.
6. From the curves, determine the time to peak and the unit hydrograph peak for the degree of development involved.
7. Develop the unit hydrograph. We have studies under way to develop curves for hydrograph width for base, 50 percent of peak and 75 percent of peak, but these have not been completed as yet. Preliminary

studies indicate that these relationships will be similar to those of the Corps of Engineers. Presently, however, the hydrograph shape is based on the shape of recorded hydrographs for similar degrees of development, with the peak and time to peak defined by the curves.

In order to develop simple design criteria for Houston, we utilized these design procedures to prepare synthetic unit hydrographs for about 50 drainage areas of various sizes and shapes. Storm hyetographs such as that shown in figure 13 were developed based on recorded storm data and utilizing procedures similar to those given in the Chicago Hydrograph method. These design storm rainfall rates were applied to the unit hydrographs to develop storm hydrographs.

The peak discharges thus computed were plotted as a function of drainage area, giving the bands shown in figure 14. For design purposes in the Houston area, it is considered suitable to design small storm sewers and laterals for a two to three-year storm. Because of the larger areas subject to flooding from the larger sewers and ditches, the desirable design frequency increases to five and ten years for the large sewers and ditches and to 25 years for bayous tributary to main outlet channels. This changing frequency concept is illustrated in figure 14.

Figures 15, 16, and 17 show the drainage area versus discharge curves which have been adopted for Houston for design of sewers and ditches up to 20 square miles. For larger areas--actual detailed hydrograph studies are used. For small areas, two curves are used to account for higher runoff rates from commercial areas, such as the downtown area, and from large paved areas such as major regional shopping centers and the Astrodome with its large

parking areas. While these curves appear deceptively simple--and they are, of course, simple to use--they incorporate rather involved hydrologic techniques which would not be feasible for application individually to the design of small sewers and ditches.

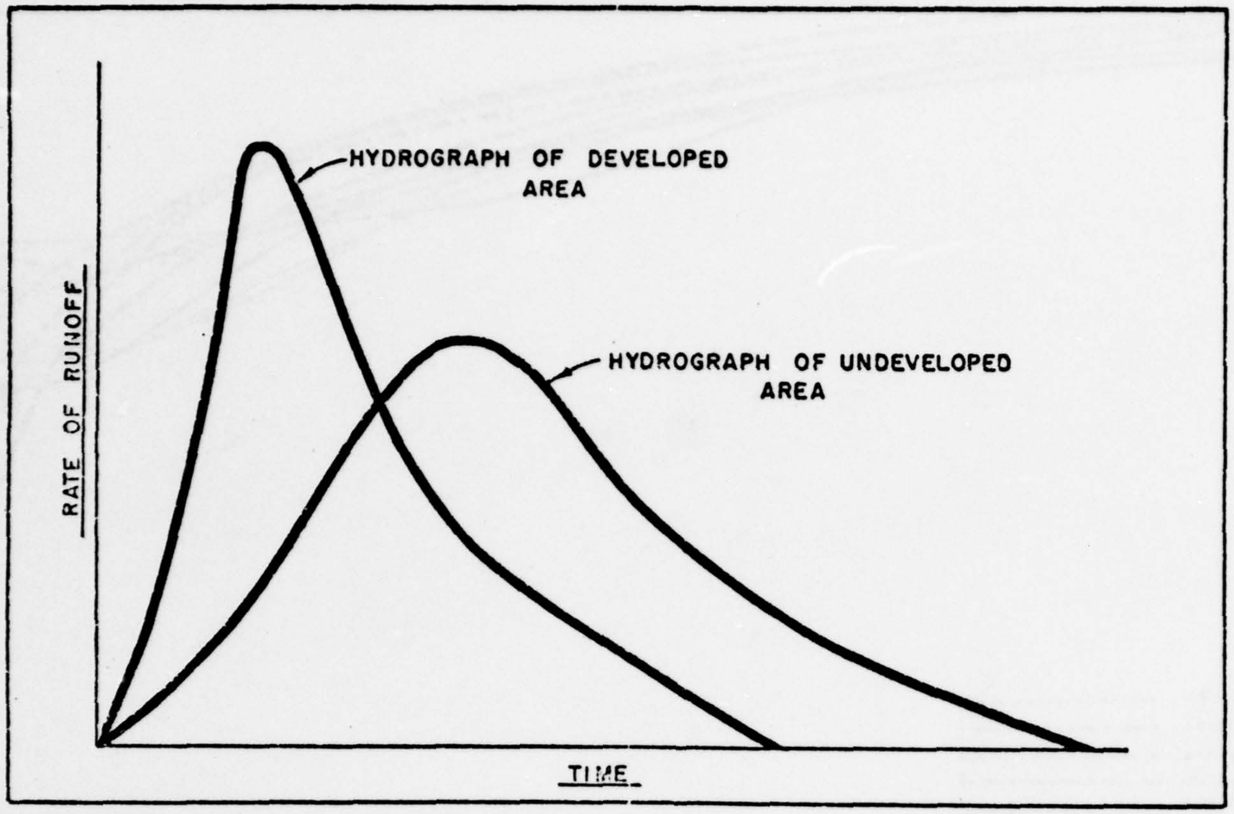
We do not suggest that the design curves for Houston are necessarily applicable in other areas. We do, however, feel that this design concept has merit in providing simple design techniques based on the most recent hydrologic concepts. We feel that the techniques using basin factor relationships are applicable to other areas and may represent one of the simplest and most convenient means of estimating the effects of future urban development in the design of drainage and flood control facilities.

It has been clearly demonstrated in the Houston area, and corroborated by similar reports from other areas, that urbanization of an area will increase peak discharge rates for a given storm by a factor of from two to three times. Furthermore, times of runoff are greatly reduced.

This becomes of vital importance in the design of flood control improvements in urban areas, and in evaluating the extent of future flooding in flood plains. In hurricane flood protection projects, where storm runoff must often be pumped, increasing urban development can result in very significant increases in pumping capacity.

It is considered that VanSickle has made a significant contribution by development of a method by which runoff from urban areas may be analyzed relative to the degree of urbanization of the watershed. It is possible that his methodology will find application in the Corps of Engineers' area of interest in the field of urban hydrology. As more data becomes available

from rainfall-streamflow gaging networks in our urban areas, we hope to become better able to evaluate the effects of urbanization on total runoff.



Effects of watershed development-storm hydrographs

Figure 1 Effects of Watershed Development

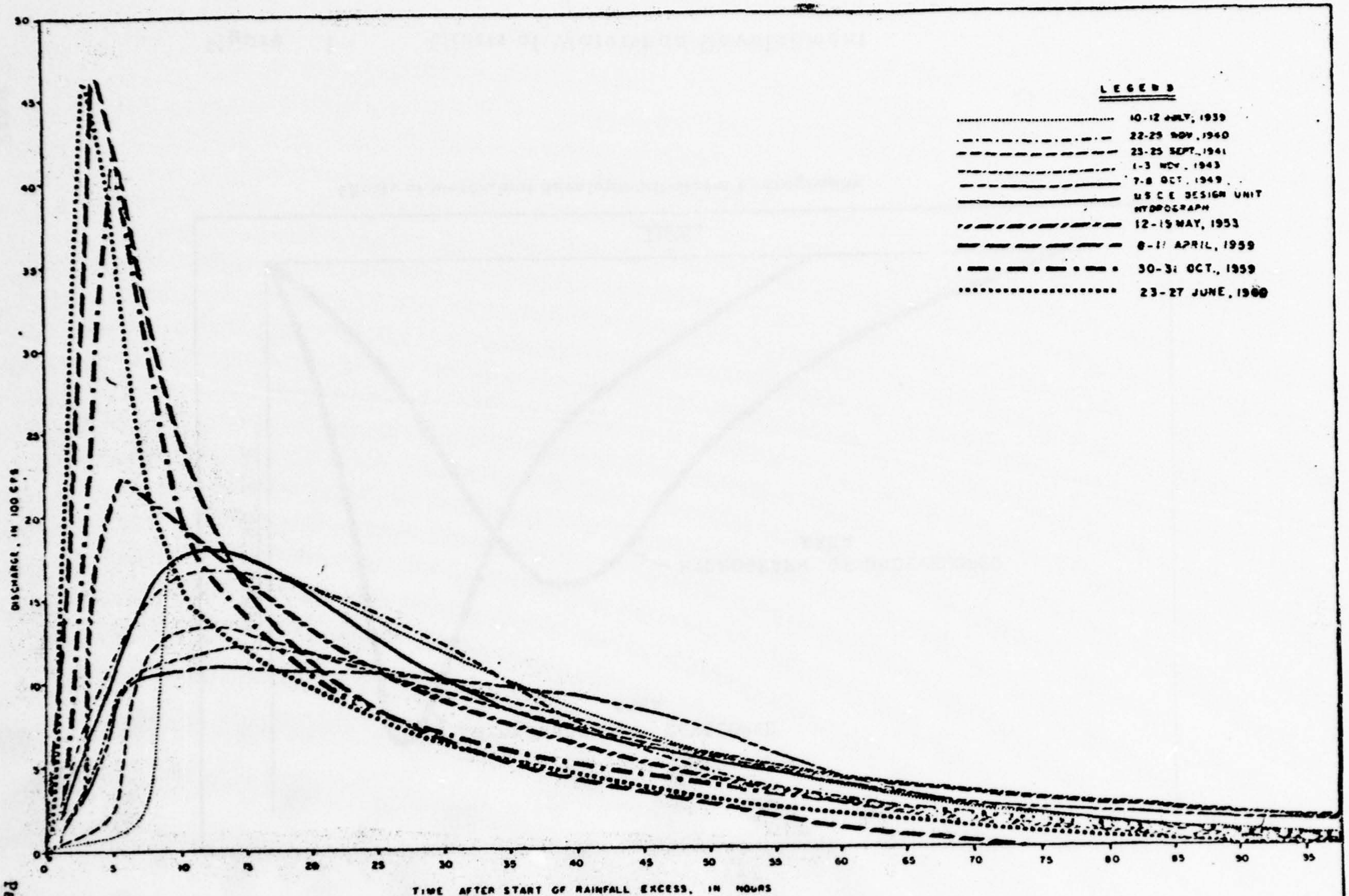


FIGURE 2 BRAYS BAYOU UNIT HYDROGRAPHS, 1939-1960, SHOWING EFFECTS OF WATERSHED DEVELOPMENT.

UNIT HYDROGRAPH CHARACTERISTICS

Q_p	q_{pR}	t_{pR}	C_t	C_p640	$\frac{C_p640}{C_t}$
2407	26.16	9.00	1.41	235	167
2278	24.76	7.85	1.22	195	160
1388	15.09	8.07	1.17	123.7	106
2231	24.25	11.57	1.67	286	172
1893	20.58	8.05	1.17	168	144

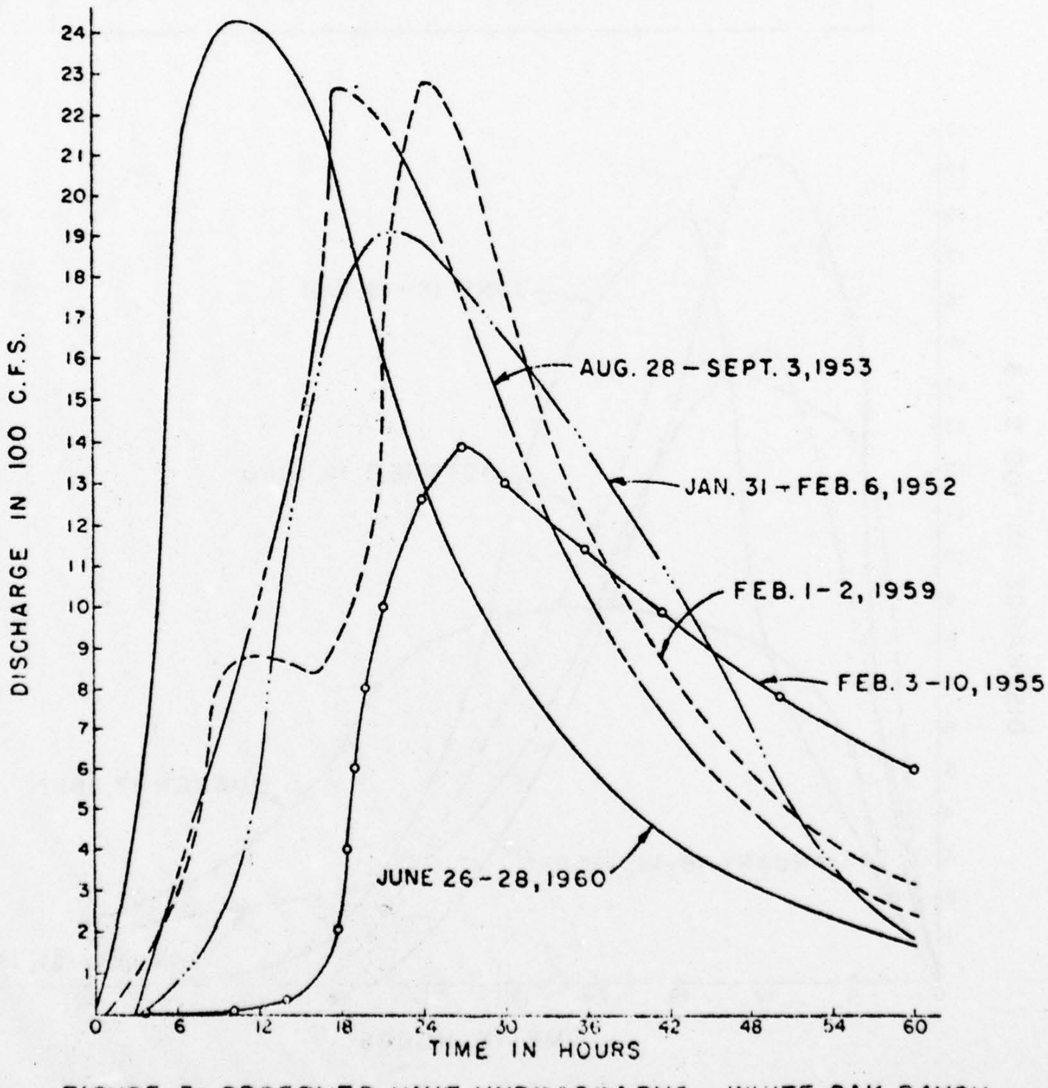


FIGURE 3: OBSERVED UNIT HYDROGRAPHS – WHITE OAK BAYOU

UNIT HYDROGRAPH CHARACTERISTICS

Q_p	q_{pR}	t_{pR}	C_t	$C_p 640$	$\frac{C_p 640}{C_t}$
1798	31.00	13.75	2.57	426	166
1456	25.14	8.00	1.49	201	135
893	15.40	13.08	2.44	201	83
1481	25.53	12.70	2.37	324	137
1933	33.33	9.00	1.68	300	180

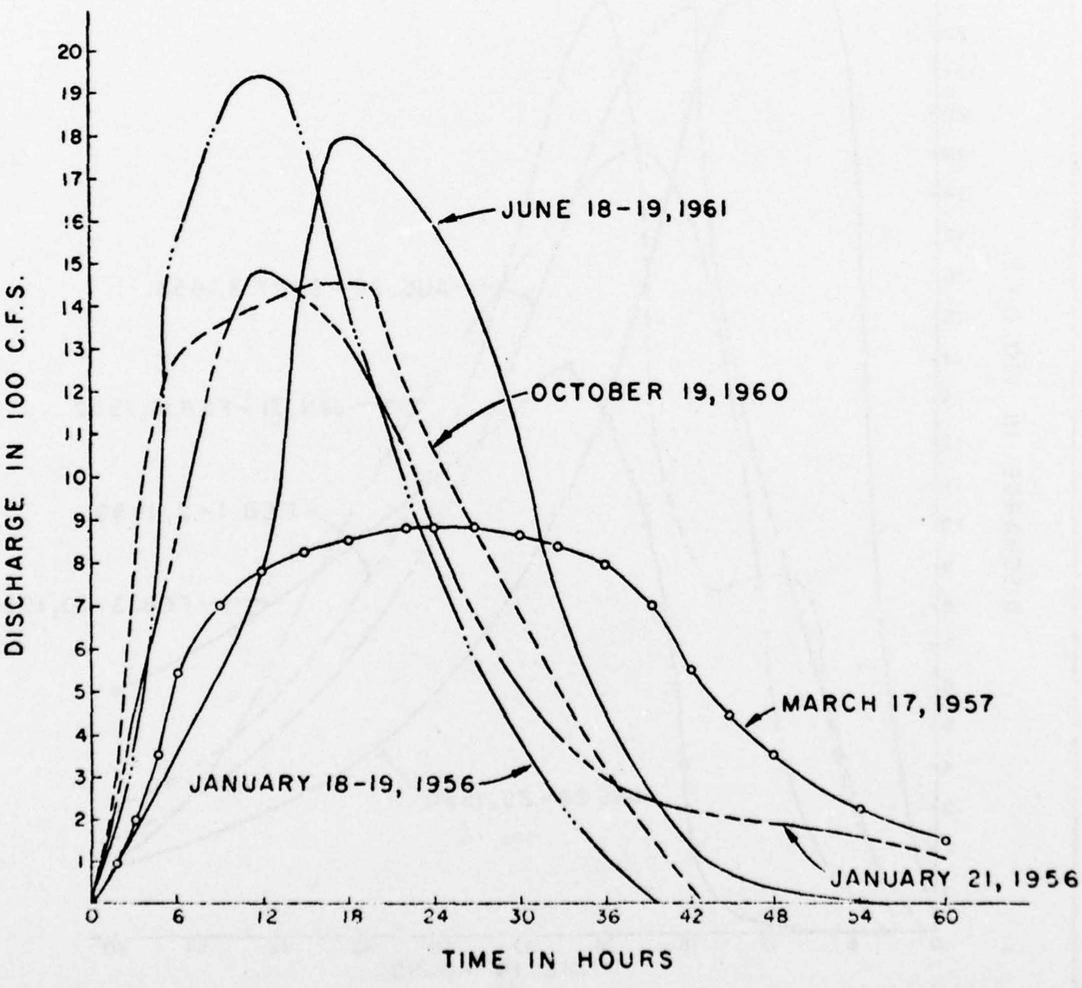


FIGURE 4: OBSERVED UNIT HYDROGRAPHS — BUFFALO BAYOU

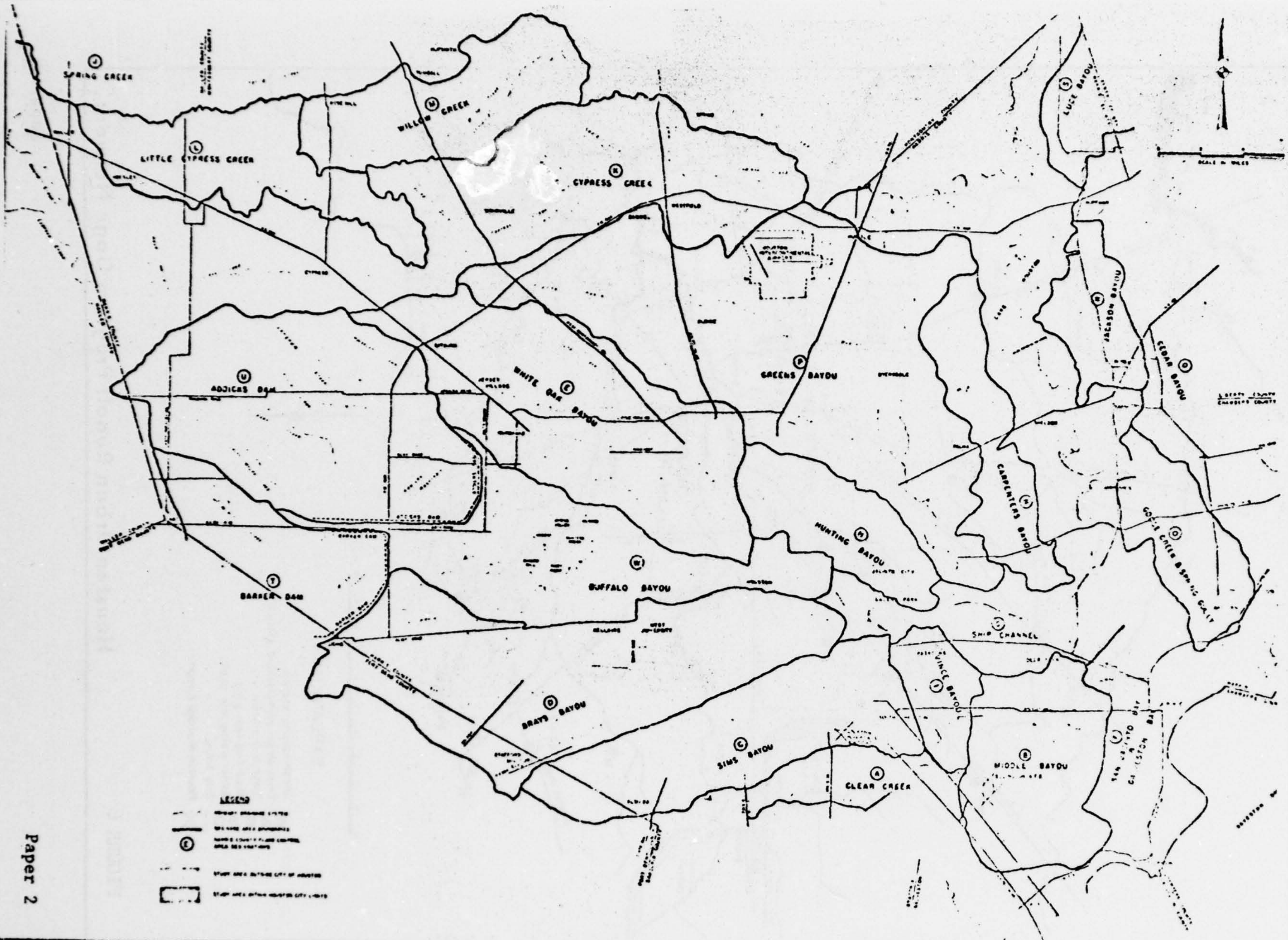


FIGURE 5 LIMITS OF STUDY AREA - COMPREHENSIVE STUDY OF DRAINAGE FOR METROPOLITAN HOUSTO

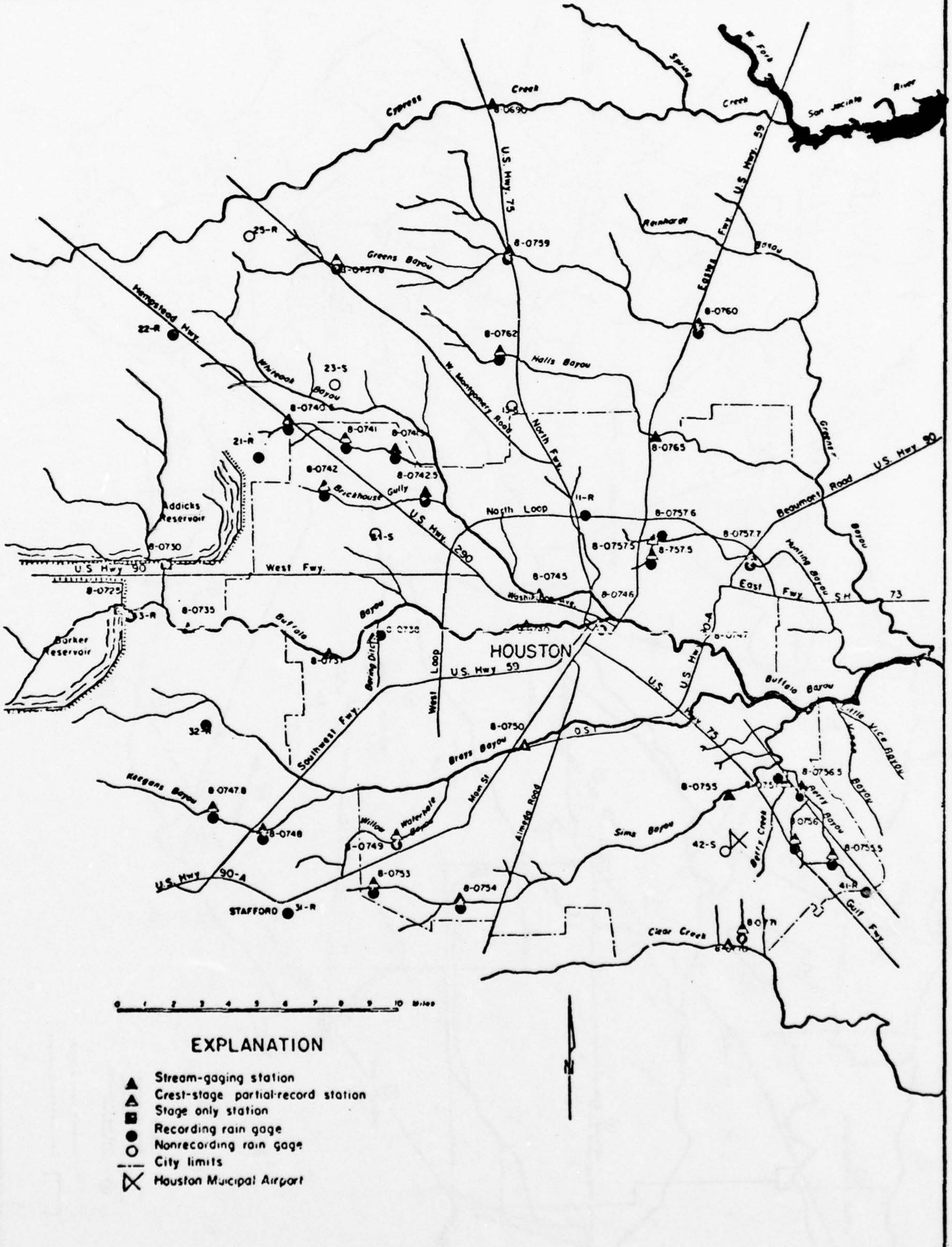


FIGURE 6 Houston Urban Runoff Program Gage Network

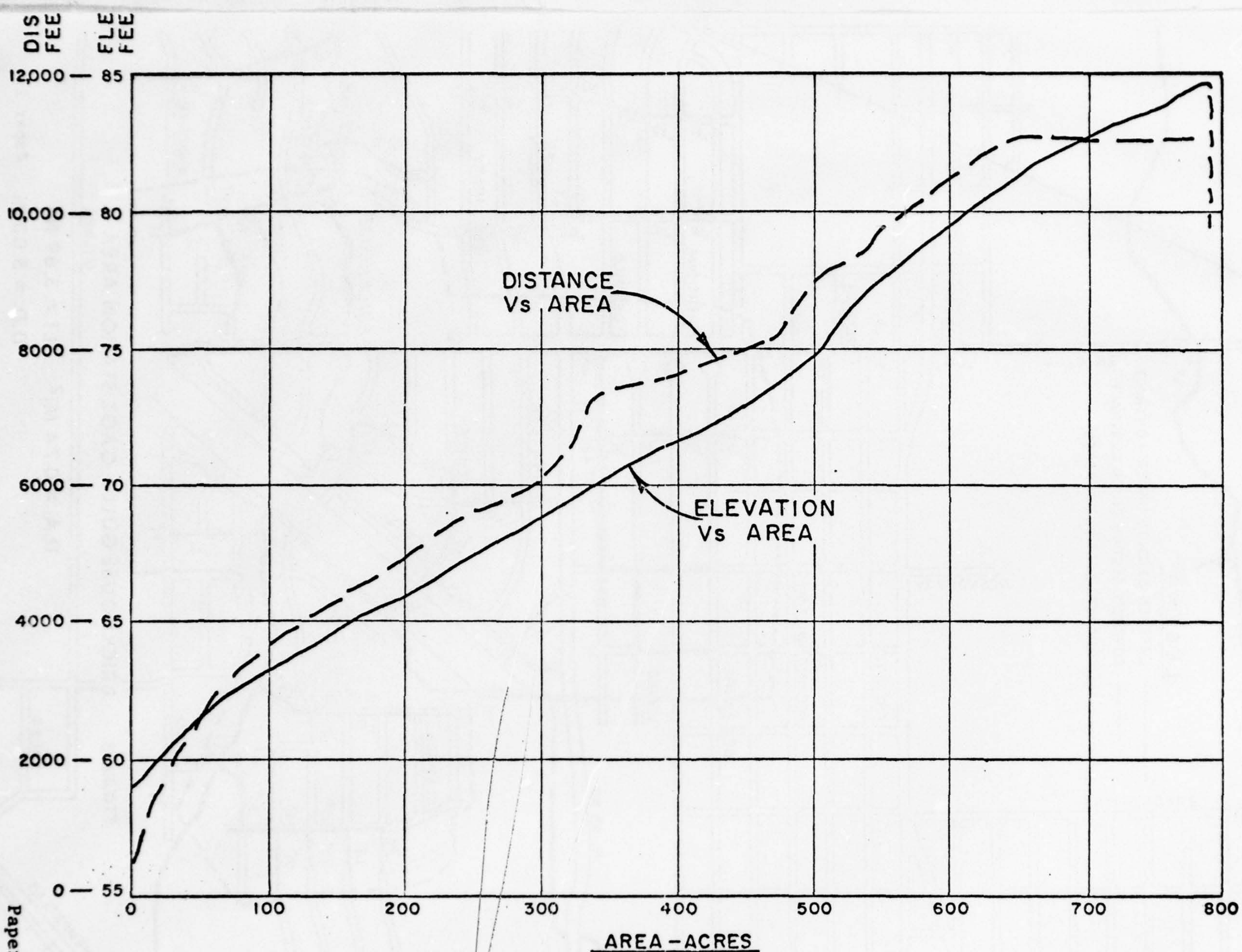


FIGURE 7 **TYPICAL LENGTH - AREA AND ELEVATION - AREA CURVES**

FIGURE 8 BRICKHOUSE GULLY GAGE STATION AREA

D.A. = 0.74 Mi^2 LT = 3.69 Mi

D.D. = 5.00Mi

Paper 2

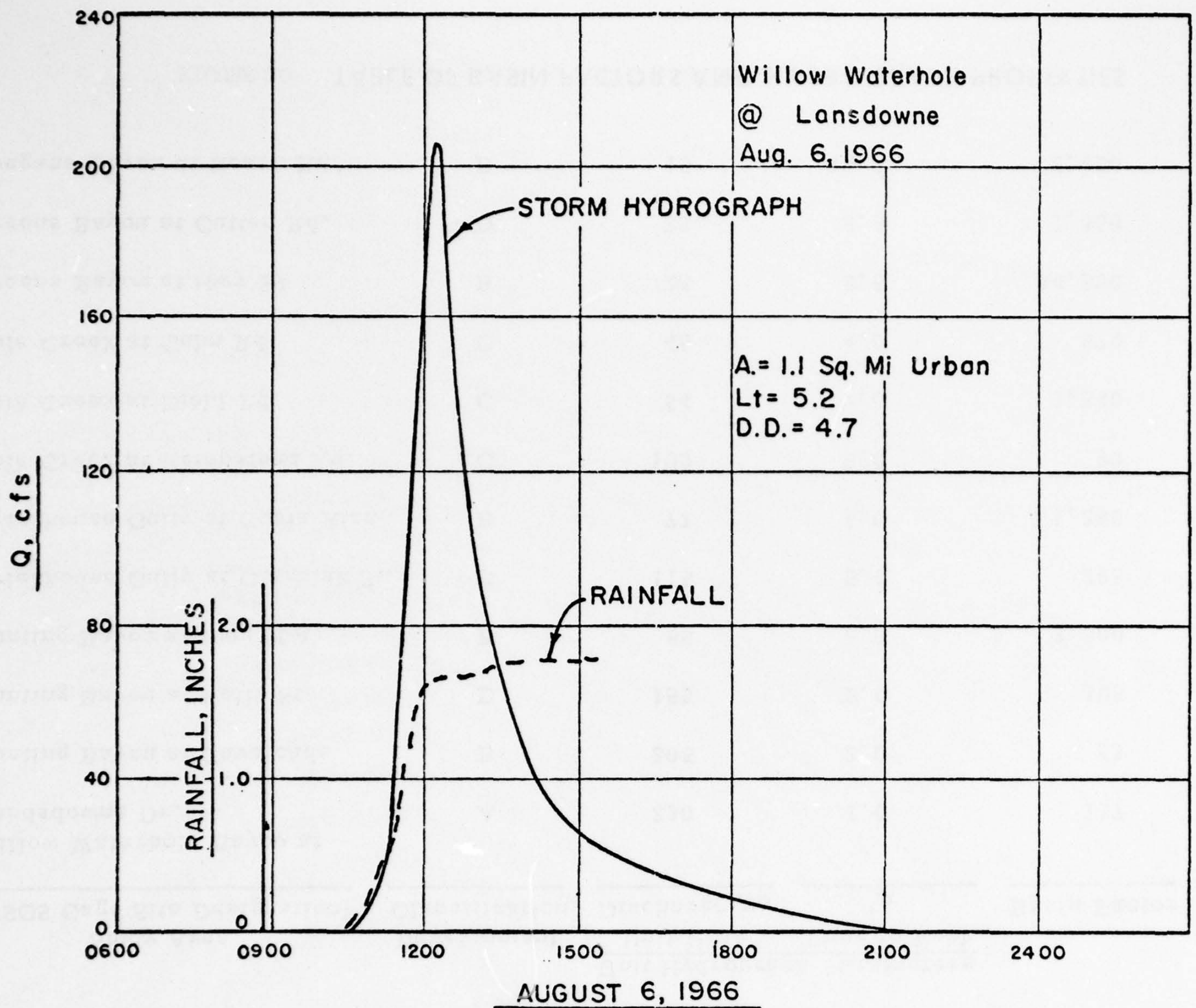


FIGURE 9 **WILLOW WATERHOLE BAYOU - STORM RUNOFF HYDROGRAPH**

Table II

Summary of Data

Study Area (USGS Gage Site Designation)	Development Classification	Unit Hydrograph Parameters: Unit Peak Discharge, q_p	Unit Hydrograph Parameters: Time to Peak t_p	Basin Factor
Willow Waterhole Bayou at Landsdowne Dr.	A	230	1.0	117
Hunting Bayou at Cavalcade	B	205	2.0	53
Hunting Bayou at Falls St.	B	155	2.0	105
Hunting Bayou at Hwy 90A	B	55	6.0	3,800
Brickhouse Gully at Clarblak St.	B	115	5.0	285
Brickhouse Gully at Costa Rica .	B	77	6.0	1,280
Cole Creek at Hempstead Rd.	C	100	5.0	90
Cole Creek at Diehl Rd.	C	64	7.0	1,240
Cole Creek at Guhn Rd.	C	46	4.0	870
Greens Bayou at Hwy 59	B	28	8.5	34,500
Greens Bayou at Cutten Rd.	D	23	8.5	1,360
Keegans Bayou at Roark Rd.	D	15	14.5	2,450

FIGURE 10 **TABLE OF BASIN FACTORS AND HYDROGRAPH PROPERTIES**

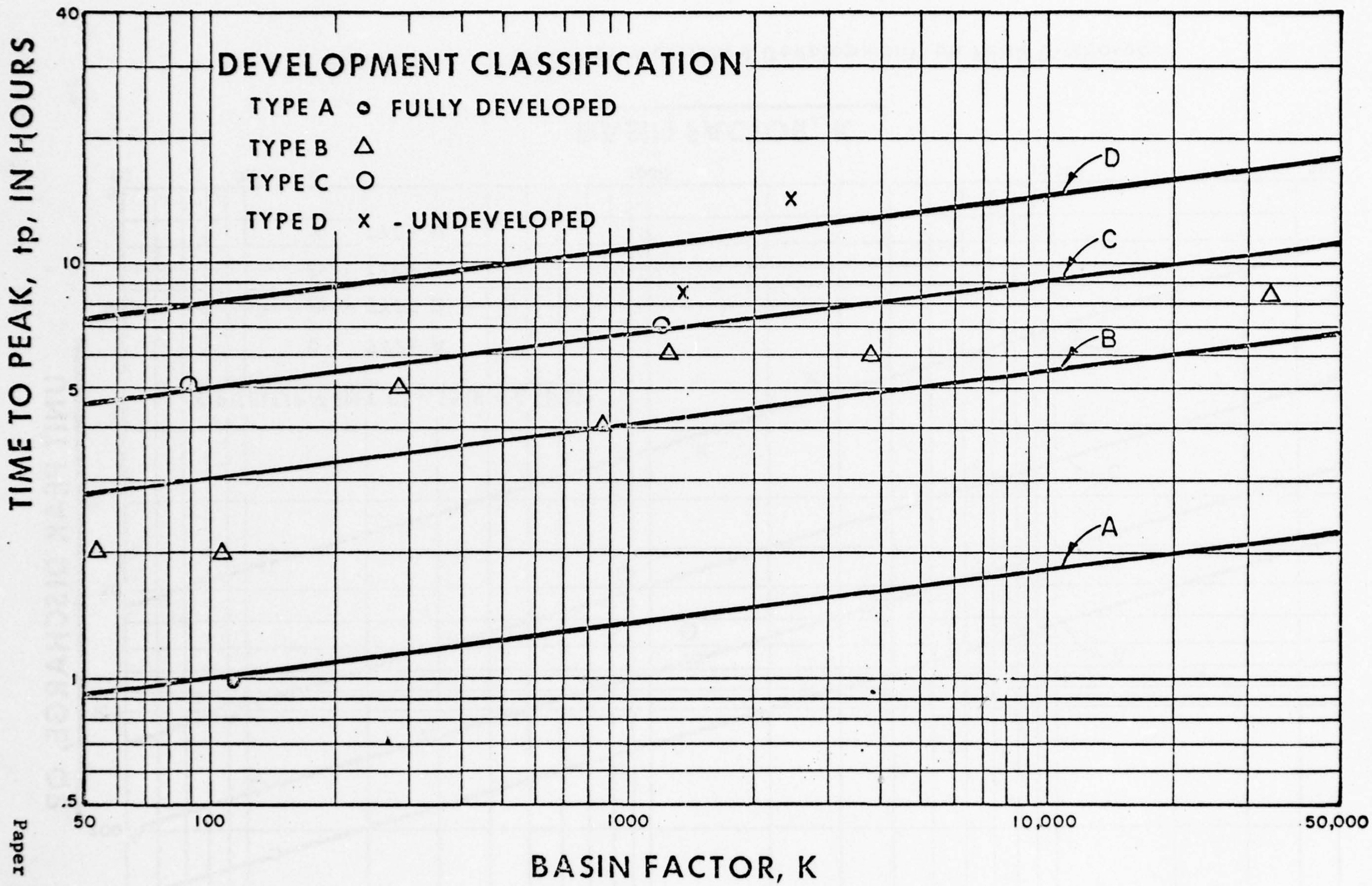


FIGURE 11 Effects of Watershed Development on Time to Peak

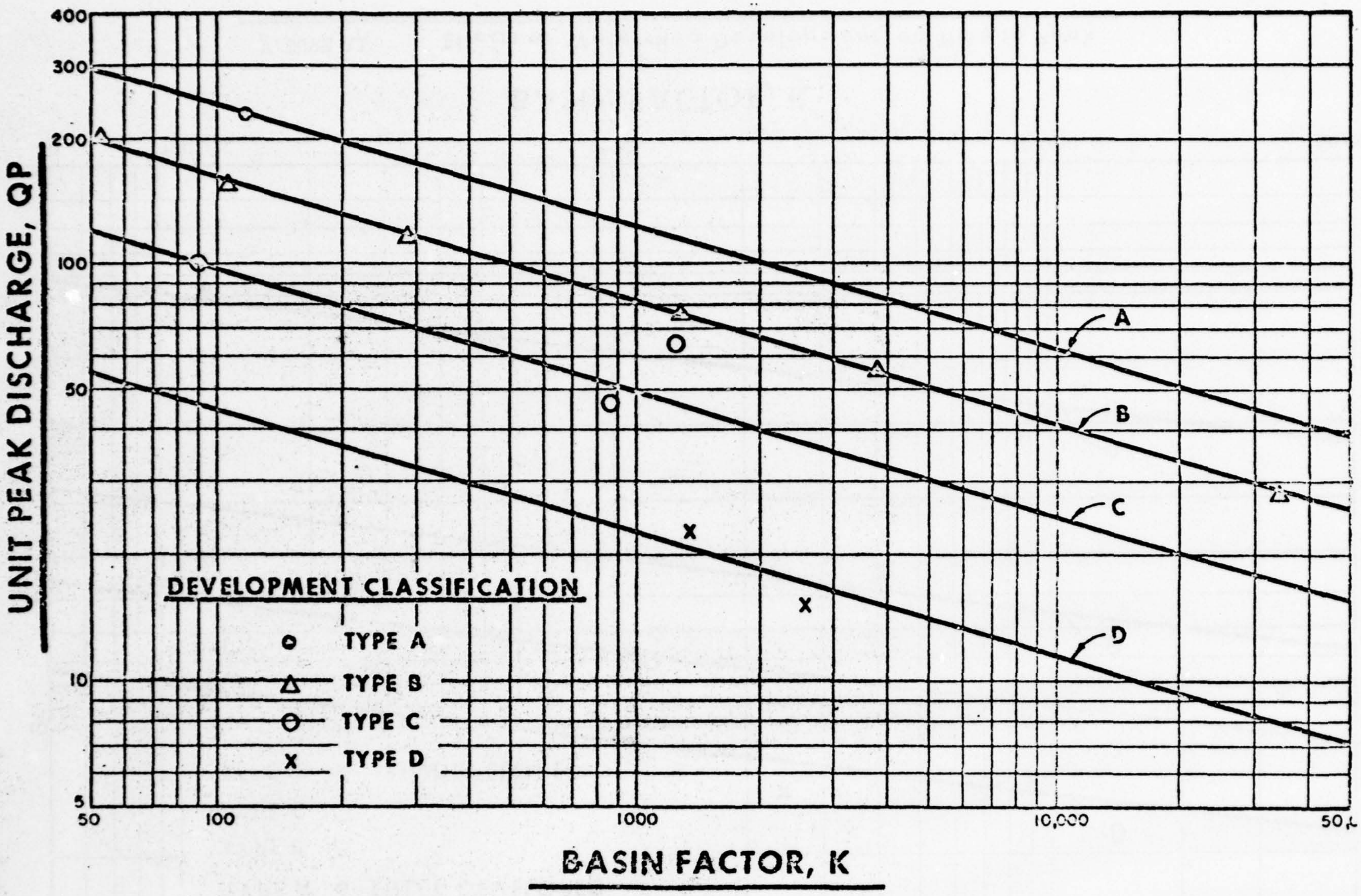


FIGURE 12 Effects of Watershed Development on Peak Discharge

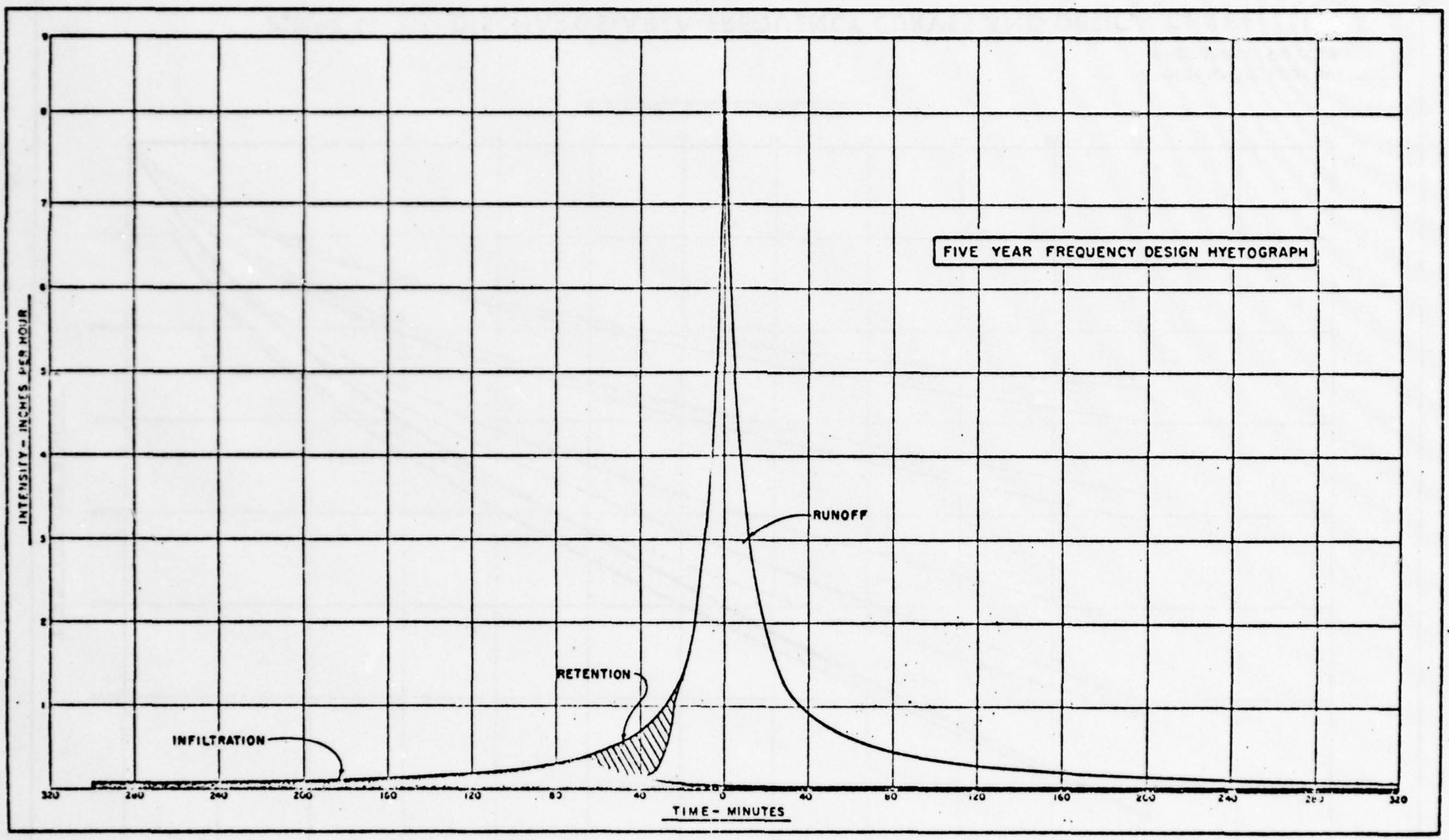


FIGURE 13 TYPICAL DESIGN STORM HYETOGRAPH

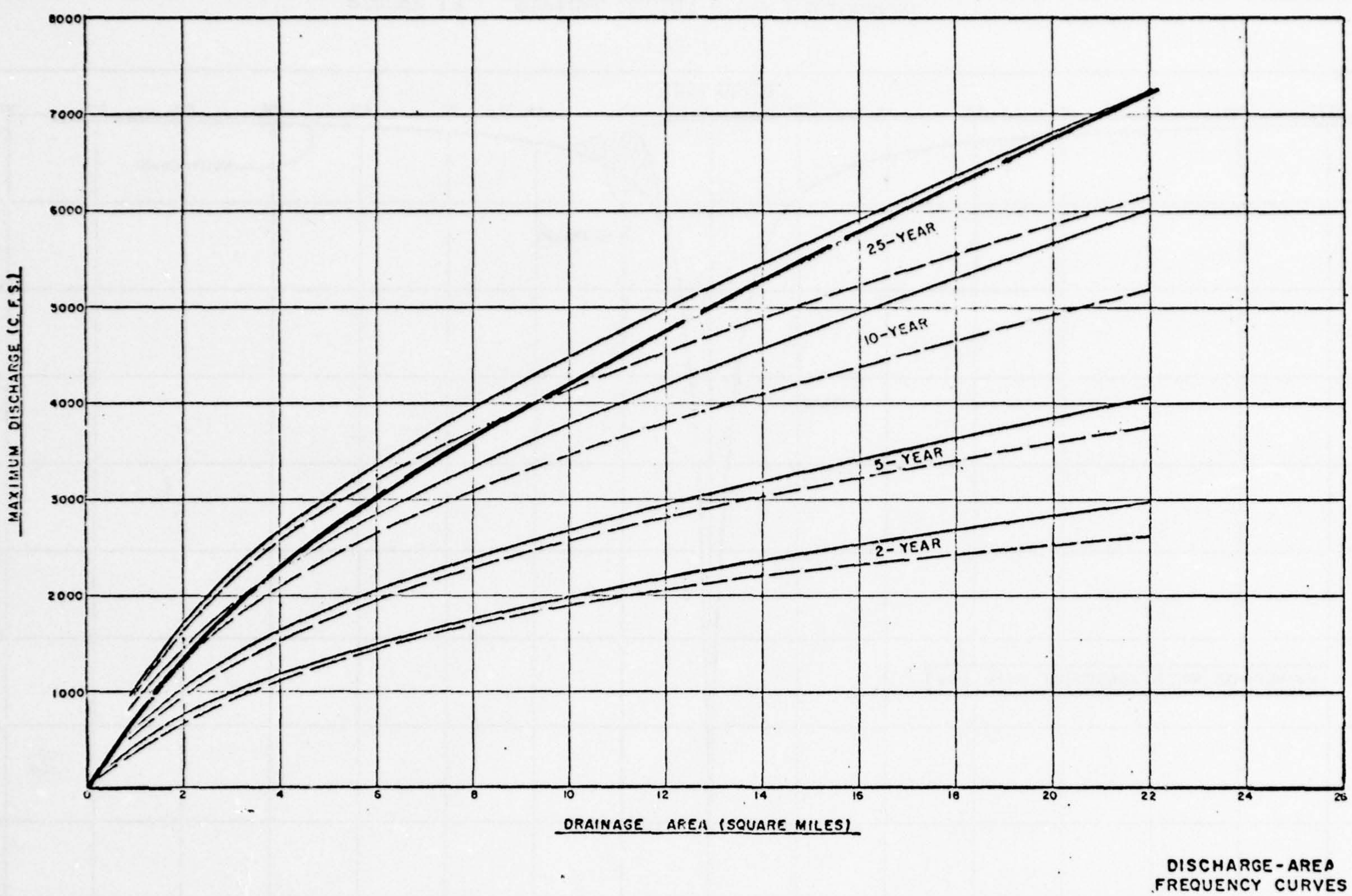


FIGURE 14 DISCHARGE-AREA-FREQUENCY CURVES AND DESIGN CURVE

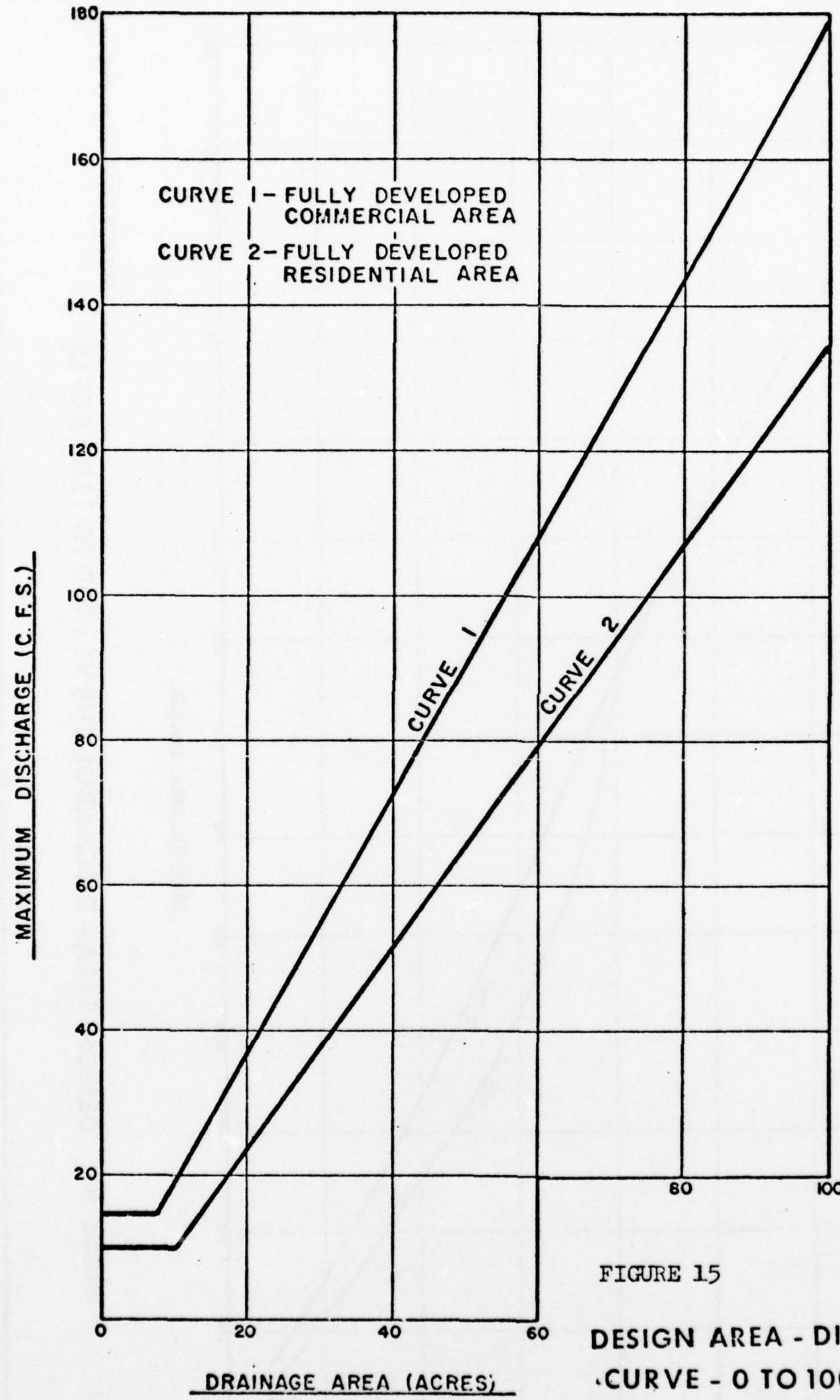
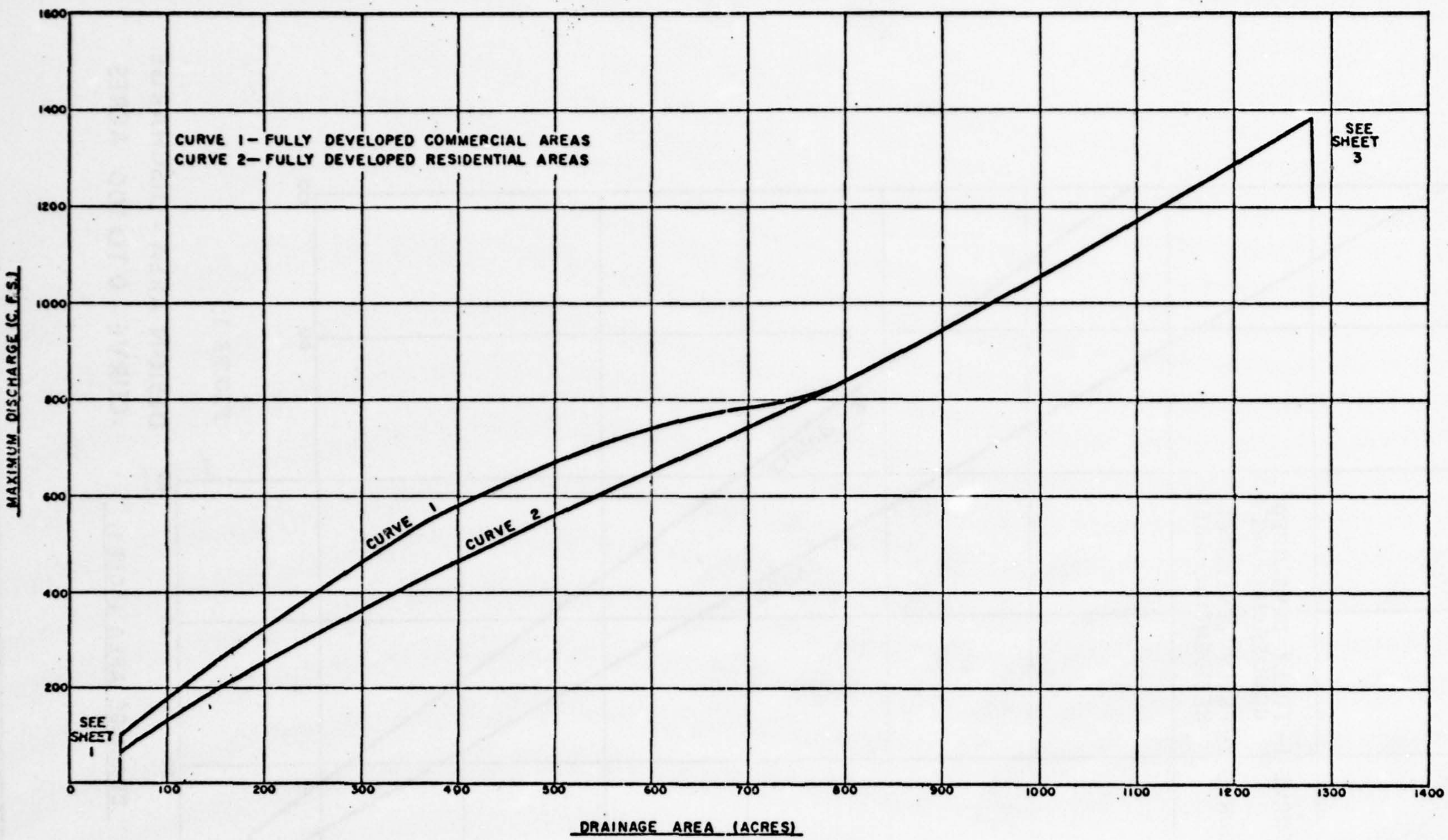


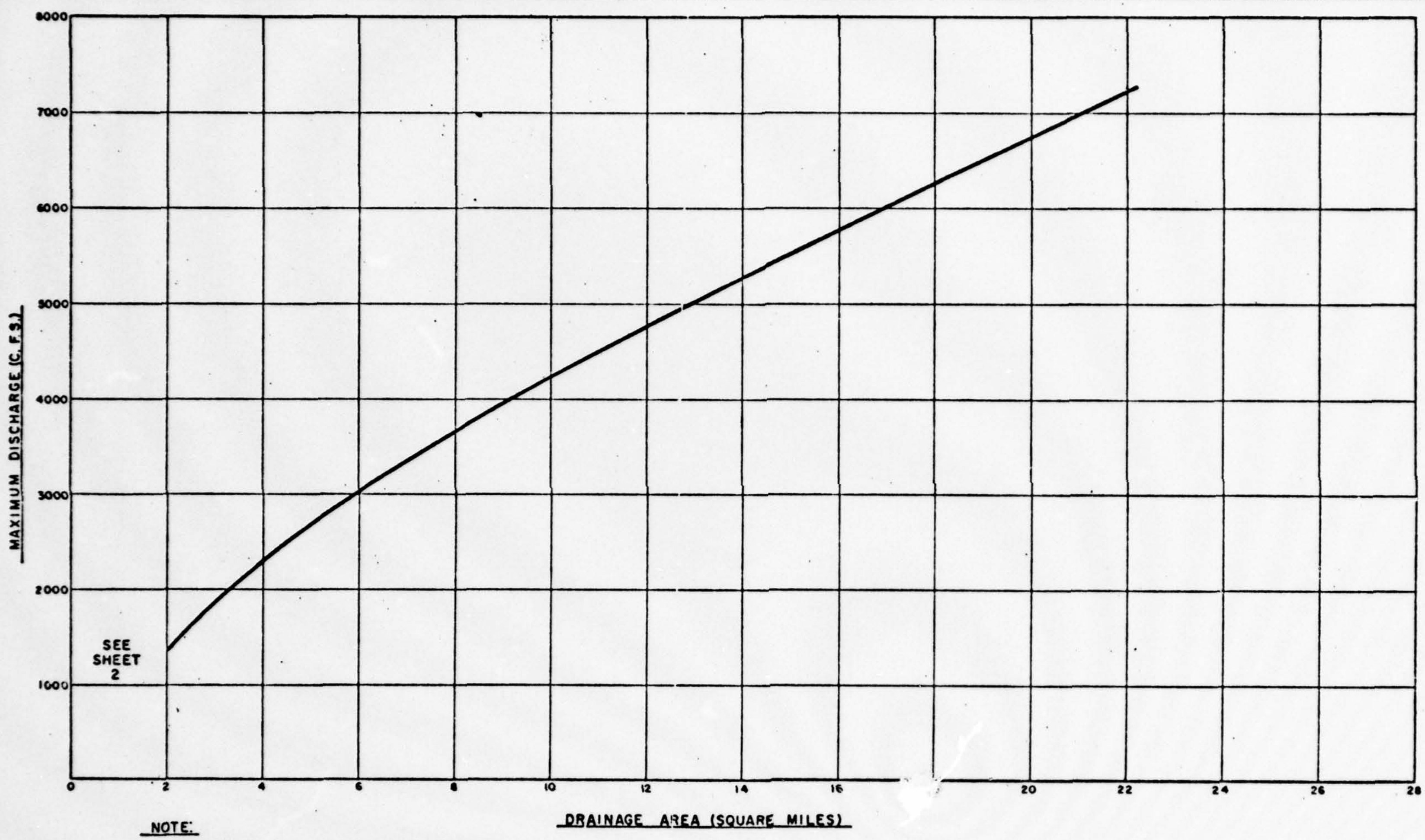
FIGURE 15

DESIGN AREA - DISCHARGE CURVE - 0 TO 100 ACRES



AREA DISCHARGE CURVE

FIGURE 16 DESIGN AREA - DISCHARGE CURVE - 100 TO 1200 ACRES



NOTE:
DISCHARGES SPECIFIED BY CURVE SHOULD BE CONSIDERED DESIGN DISCHARGES CORRESPONDING TO DESIGN WATER SURFACES IN DITCHES AND NOT BANK-FULL WATER SURFACES.

AREA DISCHARGE CURVE

FIGURE 17 DESIGN AREA - DISCHARGE CURVE - 2 TO 22 Sq MILES

EFFECTS OF URBAN DEVELOPMENT
ON STORM RUNOFF RATES

Discussion

Question, Mr. Jones: How is the drainage density estimated? What is involved in the selection of a drainage density for ungaged areas outside the Houston area? It appears that a study of a gaged area in the vicinity of a project study should be made to establish criteria for selecting drainage densities before this method could be used in other parts of the country.

Reply, Mr. Hare: Drainage density may be determined by two ways: (1) by actual analysis of an existing system or (2) by a design analysis of a system that would be required for a completely developed area. Each area would require a separate study as no two widely separated areas would have identical requirements. Possibly a typical density pattern could be derived for each area by analyzing existing drainage patterns in major urban areas.

Question, Mr. Childs: Figure 2 shows wide deviation in timing and peaks of unit hydrograph with urban growth. Should the basin be divided into two sub-areas: one with a unit graph reflecting the rapid urban runoff; and the second for the slower runoff from the upstream undeveloped area?

Reply, Mr. Hare: Subdivision of the area would be an acceptable approach to the problem. If adequate gage records were available, we could analyze the area as two separate watersheds. However, the results of our study are representative of the total watershed above the gaging station. In this case, the percentage of the watershed not devoted to urban pursuits is not significant.

Question, Mr. Beard: Is it generally assumed and is it generally true that runoff computed from storm rainfall of a specific frequency has essentially the same exceedence frequency?

Reply, Mr. Hare: We have assumed this to be generally true; however, when developing synthetic discharge-frequency curves, we attempt to adjust rainfall losses with the frequency of the storms in order to approximate a curve comparable to a discharge-frequency curve for similar area with stream-gage records. Floods resulting from various frequency rainstorms will not necessarily produce floods of identical frequency; rather, the magnitude of the flood appears to depend more upon antecedent moisture conditions, current channel conditions and other factors subject to unpredictable changes, especially for the more frequent events.

Question, Mr. Beard: Has any thought been given to developing a rainfall-runoff model that would account for areal differences in rainfall patterns? Might such a model explain the apparent erratic variations among the unit hydrographs shown in Figure 4?

Reply, Mr. Hare: I feel that much of the variation encountered in the Buffalo Bayou study resulted in non-uniformity of the areal distribution for the selected storms. The area is rather large and rainfall data from recording gages may not be adequate. At present, we have not attempted to develop any type of rainfall-runoff model to analyze effects of non-uniform rainstorm. This would be an excellent approach to solving some of our urban runoff problems.

Question, Mr. Clark: Have you compared the "Van Sickle" method with say Cartor's method as shown in the USGS open file report, "Effects of Urban Development on Floods"?

Reply, Mr. Hare: We have made no comparisons as of present and have no plans to do so. However we may do so in the future as we analyze more of the available data.

Comment, Mr. Northrop: The "K factor" concept is a good beginning on evaluation of changes in channel efficiency. It works so well here, probably, because Houston is a provice of similar geologic characteristics. Perhaps it should be termed "channel length" or "channel efficiency" rather than "Basin Factor". As in the case of Child's paper a "hydrologic" description of the basin giving stream characteristics and retention capability of the soil would help to relate your experience to other areas of the country. A summary of what is being done in the City of Houston both by Van Sickle and the Corps would be an excellent addition to the paper even if you did not get into the mechanics of developing the floods.

Reply, Mr. Hare: At this time, it is not believed to be feasible to prepare such a summary or geologic and hydrologic description. As more data becomes available, this information can be furnished as desired.

Question, Mr. Constant: On page 8 of the report, some significance is given to Van Sickle's observation that with comparable L and L_{ca} values, the peak discharge is a function of the ratio of the coefficients 640 C_p and C_t. Since 640 $C_p = q_p t_p$ and $C_t = t_p/(LL_{ca})^{.3}$ then

$$\frac{640C_p}{C_t} = \frac{q_p t_p}{t_p/(LL_{ca})^{.3}} = q_p \, (LL_{ca})^{.3}$$

If $(LL_{ca})^{.3}$ remains the same, then, he is saying the peak discharge is a function of the unit peak in cfs/sq mi. Would you comment?

Reply, Mr. Hare: The point that was made on page 8 was to the fact that the "change in ratio of coefficients more accurately describes the effects of urban development than the variations in the coefficients themselves". His point here is that coefficients might possibly be estimated for ungaged areas using the relationship described, when it may be possible to estimate one of the coefficients with a somewhat greater degree of confidence than for the other. No attempt was made to establish any relationship of the ratio other than as a method of expressing the effects of urban development or evaluating the degree of development in the watershed.

EFFECTS OF URBANIZATION ON ANNUAL
PEAK FLOW FREQUENCY ANALYSIS

by Donald L. Robey [1]

Urban hydrology is the newest and possibly the most rapidly changing field of hydrology. It was originally just concerned with the downtown paved area (i.e. storm drainage design). This concept is not sufficient today because of the current urban sprawl. Suburban areas in all parts of the nation are growing at a remarkable rate. Streets, housing developments, and shopping centers are replacing farms and woodlands. Urbanization must be considered as a characteristic of our time, but the effects of changed land use and improved drainage facilities that come with urban development on peak flow frequency analysis must be predicted.

The hydrology of urban areas is exceedingly complex and not completely understood as yet.[1] It has been seriously neglected possibly due to the peculiar nature of the gaging problem associated with urban runoff. Until recently there has been little or no reliable field data with which to evaluate the effect of urbanization on peak flow frequency.[2] An urban drainage system is highly variable in characteristics such as slope, size, shape, roughness, and degree of imperviousness. Because of this variability, most studies have been limited to small, controlled plots with the major emphasis placed on rainfall-runoff relationships.[3] The transferability of results from one area to another becomes at best, questionable.

[1] Basin Planning Branch, Baltimore District

Ideally, hydrologic studies for the design and economic analysis of flood control measures should be based on long-term stream gage records. In an urban environment, an approximate procedure must often be substituted because of the availability of adequate data.[4] An estimate of the frequency and magnitude of flooding can be made utilizing easily obtainable watershed characteristics such as slope, drainage area, etc. In a peak flow frequency analysis using historical records, an underlying assumption is made that all events are random and have occurred under similar basin conditions. In an urban drainage system, this is not the case due to drastic changes in land use and improved drainage facilities that come with urban development over a period of time. If a watershed retains a nearly uniform physical status during a period when discharge records are obtained, modern flood prediction techniques lead to a reasonably good estimate of flood frequency.[5]

The frequency of flooding is a necessary consideration in planning land use and development since as a drainage basin is changed from a natural or rural condition to a suburban or urban condition, the magnitude and frequency of flooding also changes. It is the purpose of this paper to examine relationships for estimating the magnitude and frequency of occurrence of flood peaks on a drainage basin having a high degree of urban and suburban development.

The basin used in this study is Fourmile Run located in northern Virginia. At the U.S.G.S. gage at Alexandria, Virginia, the drainage area is 14.4 square miles, and the channel capacity is approximately 2700 cfs. Fourmile Run has a total drainage area of 19.1 square miles, of which 3.2 square miles are within the City of Alexandria, Virginia, an independent community not within any county; 0.6 square miles in the independent City of Falls Church, Virginia; 13.2 square miles in Arlington County, Virginia; and 2.1 square miles in Fairfax County, Virginia. In recent years the area has undergone considerable development and redevelopment.

The climate of the basin is humid. The average annual precipitation over the basin is about 42 inches. Annual extremes in the area have ranged from about 21 to 61 inches. The average annual snowfall in the basin is about 18 inches. In general, snow remains on the ground only a few days and no significant accumulation occurs. The amount of snowmelt as well as the rainfall intensity typical of winter and spring storms are generally not associated with flooding in the small Fourmile Run Basin.[6]

The drainage area characteristics of Fourmile Run are such that flood conditions are produced by intense rainfall of short duration. This type of rainfall occurs most frequently from thunderstorms and occasionally as a result of hurricanes or

extratropical "lows". Thunderstorms or cloudbursts usually occur in the summer months as a consequence of the rapid rise of warm moist air. The area covered by such storms is small with high precipitation intensity of short duration, and is most critical for small watersheds such as Fourmile Run.[6]

The period of record for the Fourmile Run gage at Alexandria is from 1951-1969. The annual peaks for this period are tabulated below:

Water Year	Annual Peak (CFS)
1951	1,300
1952	1,600
1953	3,450
1954	854
1955	2,120
1956	1,350
1957	810
1958	1,450
1959	1,250
1960	810
1961	3,600
1962	1,060
1963	11,700
1964	1,800
1965	2,560
1966	6,900
1967	6,290
1968	5,040
1969	14,600

On 22 July 1969 the maximum flood of record with a peak discharge estimated by the U.S.G.S. at 14,600 cfs occurred on Fourmile Run. The July 1969 flood could possibly be considered an "outlier" and not representative of the 19 year period of record. A short record station such as Fourmile Run can easily have its underlying flood distribution obscured by erratic chance variations.

A recent study by the U.S.G.S. drew the following general conclusions on the effects of urban development on floods in northern Virginia:[2]

1. The lag time appears to be the parameter most affected by urbanization. The lag time for a completely storm-sewered system is about one-eighth that of a comparable natural system, while storm sewering the tributaries reduces the lag time to about one-fifth that of a comparable natural system.

2. The relationships developed indicate that urban and suburban development significantly change flood magnitudes. On small, steep basins, drainage improvements alone may triple average flood sizes, and complete development of stream channels and the basin surface may increase average floods by a factor of 8.

3. A complete impervious surface will increase the average size flood by a factor of 2-1/2, but an impervious surface has a decreasing effect upon larger floods and has an insignificant effect upon the flood of 100-year recurrence interval.

The above conclusions give support to those drawn in another U.S.G.S. study that concerned itself with effects of urbanization on flow frequency analysis.[7]

The U.S.G.S. study in northern Virginia developed the following relationships using multiple regression techniques for computation of lag time (T) and mean annual flood ($\bar{Q}$):

$$T = 0.9\ (L/\sqrt{S})^{0.5} \qquad (1)$$

where: T = lag time in hours, average time interval between the centroid of rainfall excess and the centroid of direct runoff.

L = stream length, distance in miles along the primary water course from the basin mouth (stream gaging site) to the basin boundary.

S = index of basin slope, average slope, in feet per mile, of the main water course between points 10 and 85 percent of the length (L) upstream from the stream gage.

$$\overline{Q} = 230\ KA^{0.82}T^{-0.48} \qquad (2)$$

where: $\overline{Q}$ = mean annual flood in cfs

K = coefficient of imperviousness formulated as

$K = 1.0 + 0.015I$ (I = Percent impervious).

A = drainage area in square miles.

T = lag time as defined by equation 1.

Data needed for computation of the mean annual flood ($\overline{Q}$) at the Fourmile Run gage were obtained from readily available maping of the study area (L = 7.8 miles, S = 42.5 ft./mi., I = 30 percent, and A = 14.4 sq. mi.). Using equation 1, the lag time (T) was computed as 1 hour, and the mean annual flood ($\overline{Q}$) was computed, using equation 2, to be 2,830 cfs.

A regional peak flow frequency analysis had previously been performed for the area of metropolitan Washington, D. C. that included the Fourmile Run basin. The technique used is described in the Hydrologic Engineering Center, Corps of Engineers, Computer Program Number 23-C*-L268, "Regional Frequency Computations." The Fourmile Run gaging station was dropped from the study due to poor correlations with other stations. The study did produce a regional skew coefficient for the area of +0.40, and a regional standard deviation of 0.25. Both regional values were computed as a weighted average based on extended record lengths of stations in the study area. Some of the streams included in the regional frequency analysis could be classified as being in an urban and/or suburban state. The use of these regional statistical parameters in a frequency analysis of annual peak flows in the Fourmile Run basin, is considered an acceptable possibility. The reliability of the statistics computed at any gaging station is a function of the length of record at that station. The longer the record the more reliable are the statistics. With only 19 years of record at the Fourmile Run gage, and the effects of urbanization over the period of record, the use of statistics computed from the historical record to develop a peak flow frequency curve seems questionable.

In order to demonstrate the variability of results obtained when using different approaches to define a peak flow frequency curve for the Fourmile Run basin, the following methods are described along

with computed discharges for the 25, 50, and 100-year floods for each method:

Method No. 1: Procedures as recommended by Beard[8] and the Water Resources Council[9] were used to compute a frequency relationship for Fourmile Run at the gage. For the historical record, the following statistics and discharges for the selected recurrence intervals were computed. No record adjustments were made.

			PEAK DISCHARGE IN CFS		
	MEAN(M)	= 2,380 cfs	25-Year	50-Year	100-Year
(a)	STANDARD DEVIATION(S)	= 0.39			
	SKEW COEFFICIENT(G)	=+0.67 (computed)	13,700	20,250	29,360

Using a skew coefficient of 0.0, the following peak discharges were computed:

		PEAK DISCHARGE IN CFS		
	M = 2,380 cfs	25-Year	50-Year	100-Year
(b)	S = 0.39			
	G = 0.0 (Assumed)	11,370	14,910	19,030

Using the previously computed regional skew coefficient, the following results were obtained:

		PEAK DISCHARGE IN CFS		
	M = 2,380 cfs	25-Year	50-Year	100-Year
(c)	S = 0.39			
	G =+0.40 (Regional)	12,770	17,970	24,690

Method No. 2: Procedure used is the same as in Method No. 1, except the 1969 peak was deleted from the record. Computations were made using computed statistics, an assumed skew coefficient of 0.0, and a regional skew coefficient of +0.40. Results are shown below:

		PEAK DISCHARGE IN CFS		
		25-Year	50-Year	100-Year
(a)	M = 2,150 cfs S = 0.35 G =+0.66 (computed)	10,280	14,560	20,240
(b)	M = 2,150 cfs S = 0.35 G = 0.0 (assumed)	8,740	11,137	13,860
(c)	M = 2,150 cfs S = 0.35 G =+0.40 (regional)	9,690	13,160	17,500

Method No. 3: An attempt was made to adjust the historical record at the Fourmile Run gage to alleviate the effect of changing land use in the basin. It was assumed that the years of record from 1961 - 1969 were representative of present basin conditions and adjustments were not made to these annual peaks. Using the previously mentioned U.S.G.S. generalized equations, the flows from 1951 - 1955 were increased by 48 percent according to the ratio of mean annual floods obtained from computed present conditions versus those for 1951 - 1955. The peak flows during 1956 - 1960 were increased by a variable factor assumed to decrease from 40 percent in 1956 to 8 percent in 1960. Using these adjusted peaks, a frequency analysis as described in Method No. 1 was performed. Results are shown below:[6]

		Peak Discharge in CFS		
		25-Year	50-Year	100-Year
(a)	M = 2,780 cfs S = 0.36 G =+0.55 (computed)	13,440	18,830	25,870
(b)	M = 2,780 cfs S = 0.36 G = 0.0 (assumed)	11,650	14,920	18,650
(c)	M = 2,780 cfs S = 0.36 G =+0.40 (regional)	12,950	17,690	23,650

<u>Method No. 4</u>: Procedure used is the same as in Method No. 3, except the 1969 peak was deleted from the record. Results are shown below:

		Peak Discharge in CFS		
		25-Year	50-Year	100-Year
(c)	M = 2,780 cfs S = 0.36 G =+0.48 (computed)	10,250	13,730	18,040
(b)	M = 2,540 cfs S = 0.32 G = 0.0 (assumed)	9,160	11,440	13,970
(c)	M = 2,540 cfs S = 0.32 G =+0.40 (regional)	10,070	13,320	17,290

Method No. 5: The mean annual flood ($\overline{Q}$) at the Fourmile Run gage was computed as 2,830 cfs using the U.S.G.S. generalized equations. To show the sensitivity of a frequency analysis to changes in standard deviation, the following computations were made assuming a mean annual flood of 2,830 cfs and a log-Pearson Type III distribution as referenced in Method No. 1.

		PEAK DISCHARGE IN CFS		
	M = 2,830 cfs	25-Year	50 Year	100-Year
(a)	S = 0.25 (regional)			
	G =+0.40 (regional)	8,350	10,390	12,740
	M = 2,830 cfs			
(b)	S = 0.39 (with 1969 flood)	15,300	21,540	29,600
	G =+0.40 (regional)			
	M = 2,830 cfs			
(c)	S = 0.35 (without 1969 flood)	12,870	17,490	23,270
	G =+0.40 (regional)			

As can be seen from the above approaches, there is a large variability in results when different methods are used to define a peak flow frequency relationship for a basin such as Fourmile Run. For the 25-year flood, the discharge ranged from 8,350 to 15,300 cfs, for the 50-year flood the discharge ranged from 10,390 to 21,540 cfs, and for the 100-year flood the discharge ranged from 12,740 to 29,600 cfs. The method adopted as preferred must have

some basis for selection. This basis for selection cannot be based on rigorous statistical grounds. If the years of record at the Fourmile Run gage all occurred under similar basin conditions, then selection of the "best" frequency method could be based on the relative conformance to the data.[10]

Based upon recommendations by the Water Resources Council,[9] the use of the log-Pearson Type III distribution, modified to include the recent U.S.G.S. study to define the mean annual flood, seems desirable. Since the skew coefficient has greater variability between samples than the mean or standard deviation, it is a less reliable estimator of a population statistic for a particular site. For this reason, a regional skew coefficient should be used for the Fourmile Run basin (i.e. G = +0.40). A standard deviation of 0.35 was adopted for the basin. This is the same as computed from the historical record with the 1969 peak deleted. Method No. 5-c corresponds to the "preferred" procedure outlined above. Therefore, a 25-year flood of 12,870 cfs, and a 50-year flood of 17,490 cfs, and a 100-year flood of 23,270 cfs is recommended by the author for adoption for the Fourmile Run basin.

Hydrologists have known for a long time that urbanization has major effects on peak flows. The analysis of this problem through research--plot studies has shown that the amount of influence on peak flows depends on the degree of urbanization and that this degree of urbanization seems best expressed as a percent of

impervious area. There is a definite need for more research in urban hydrology especially studies concerning peak flow frequency analysis. These effects of basin development have received only limited study because of the scant amount of available data. An important consideration in future studies should be the transferability of results from one basin to another.

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EFFECTS OF URBANIZATION ON ANNUAL
PEAK FLOW FREQUENCY ANALYSIS

Discussion

Question, Mr. Childs: Does the procedure (Method 5) lend itself for projecting frequency relationships for future years with further development in the basin? This is essential for the economic analysis of proposed improvements, and probably for establishing flood insurance rates.

Reply, Mr. Robey: It does through the adjustment in the percent imperviousness (I) used in computing the mean annual flood using the USGS generalized equation.

Question, Mr. W. Johnson: What factors indicated the 1969 flood may be an outlier?

Reply, Mr. Robey: Including the 1969 flood in the computation of frequency statistics would produce a curve indicating the higher flood peaks occurring more frequently than is considered likely. Based on available rainfall data, it was estimated that the recurrence interval of the storm producing the 1969 peak is approximately 50 years. If this 1969 annual peak were to be included in the frequency analysis then it would have a recurrence interval of about 30-40 years.

Question, Mr. Northrop: You mentioned that an impervious surface will increase the average size flood by a factor of $2\frac{1}{2}$, what is the likely increase for a major flood such as the 100 year flood?

Reply, Mr. Robey: Actually, as stated in the paper, urbanization might have an insignificant effect upon floods as high as 100-year recurrence interval. The greatest effect of urbanization is on the floods having recurrence intervals less than 50 years.

Comment, Mr. Northrop: A discussion of the range of errors that might be introduced for a given procedure would be helpful.

Reply, Mr. Robey: There is not enough information available to perform an error analysis of the complete procedure. The USGS computed the standard error of estimate for the computation of the mean annual flood to be -23.3 and +29.9 percent using the generalized equation presented in the paper.

A UNIFIED METHOD FOR COMPUTING PEAK DISCHARGE FROM UNGAGED URBAN AREAS FOR CORPS OF ENGINEERS STUDIES

By
William R. Henson[1]

INTRODUCTION

Until 30 years ago our principal urban hydrology problem was computing design discharges for storm drainage facilities. The method in general use to date has been the very irrational "Rational Formula." Recent rapid urbanization has increased the complexity of storm drainage facilities in the larger cities requiring open channel collector systems, supercritical flow channels, drop structures, pumping stations, and other hydraulic structures for efficient operation. The rate of urban expansion will determine the need for larger and even more complicated systems. In the U. S.,urban population of 125 million in 1960 is projected to reach 252 million by 2000. If urban population densities do not change, urban space requirements will double by the turn of the century. However, Meier, Darling, and Milton have predicted (1) that urban space requirements will triple or quadruple with doubled urban population.

Urbanization also presents the problem of greatly increased potential flood damages. For example in LRD, flood damage for a 2-year period in a partially urbanized watershed was 5 times greater than a completely rural watershed which was 6.5 times larger. The urban flood damage potential has promoted urban flood control, flood plain information, and flood insurance studies. The C of E needs a well-conceived comprehensive

[1]Chief, Hydrologic Engineering Section, Little Rock District

program in urban hydrology to meet these demands. The program should encompass both immediate and long-range problems. Currently, the most important need at the District level is a method for computing peak discharges from ungaged urban areas. This method should provide the basis for hydrologic studies required for flood plain information and flood insurance reports, and for the planning and design of urban storm drainage and flood control projects, which are the heart of the Corps urban water resources program.

Presently, peak discharges are computed using synthetic unit hydrographs developed from empirical methods developed for use in rural areas. The use of these methods in urban areas could prove disastrous, as VanSickle(2) has found that unit hydrograph peaks from urban areas may be from 2 to 5 times higher than from equivalent-sized rural watershed. Because of the emphasis on urban hydrology in the early 60's, a number of urban stream gaging programs and research studies were initiated. To date, no one method for determining peak discharges from ungaged urban areas has been adopted for general usage, although several have been proposed. The limited use of various proposed methods probably results from the inability of the researchers to prove that they are applicable over wide areas and changing urban conditions. The Corps needs a unified method and, as a leading Federal Agency in the water resources field, has the resources to develop such a method. Presently three general methods show promise for computing peak discharges from ungaged urban areas. These are physical models, synthetic unit hydrographs, and comprehensive simulation models.

This paper will present the characteristics of the three models, their advantages and disadvantages, their applicability to Corps use, and recommend that the Corps adopt one for use in developing a unified method for computing peak discharges from ungaged urban areas.

PHYSICAL MODELS

Physical models based on the laws of dynamic similarity have been used to predict prototype response to various hydraulic and hydrologic phenomena. The basic laws include the Froude Law where gravitational forces predominate, Reynolds Law where viscous forces predominate, and the Weber relation for surface tension effects. To cover a complete range of performance would require that the model be designed based on all three laws, which is physically impossible. The model is designed to eliminate the need for obeying all the laws, to get around this dilemma. For example, in Fourde modeling where the model is designed based on the Froude Law, the model must be built large enough to ignore surface tension effects and must be operated in the turbulent flow range to keep viscous forces constant. When these laws are applied to modeling a complete watershed, problems develop because overland flow is dominated by viscous forces and streamflow is dominated by gravity forces. This would require different scale parameters for the areas where either overland or streamflow dominates. As this is a physical impossibility, the use of models must be eliminated as a means of predicting peak discharges for a wide range of urban watersheds. Successful modeling is limited to those areas for which either overland flow or streamflow is the only significant factor. A specific example

of possible application in urban areas would be to predict runoff from a relatively flat paved parking area and to determine the change in the runoff hydrograph resulting from channel contractions or enlargements. The major disadvantage of modeling, which limits use more than any other factor, is the large investment required to build and operate the model, and to analyze and present the model results.

SYNTHETIC UNIT HYDROGRAPHS

The unit hydrograph has been used to compute peak discharges for rural areas since Sherman (3) developed the unit hydrograph theory in 1932. The Corps relies heavily on the method developed in 1938 by F. F. Snyder (4) to compute synthetic unit hydrographs for ungaged rural areas. However, only within the past 10 years have methods been advanced for computing synthetic unit hydrographs for urban areas. The works of Eagleson (5), Espey (6), and VanSickle (7), which relate pertinent basin parameters to the lag time and unit hydrograph peak are of especial interest for further consideration.

The stream gaging data used by Eagleson consisted of hydrographs of storm sewer flow on five heavily urbanized small drainage areas in Louisville, Kentucky. From the unit hydrographs derived from this data and the basin characteristics, Eagleson determined the following relations.

$$t_p = .067 \frac{(LL_{ca})^{0.38}}{S^{.5}}$$

$$q_p = 2.13 \times 10^5 S$$

Where: t_p is the lag time in hours from midpoint of unit rainfall excess to the peak of the unit hydrograph; L is the maximum travel distance along the main sewer in miles; L_{ca} is the distance along the main sewer to a point opposite the center of gravity of the basin; S is the mean basin slope in feet per foot; and q_p is the peak of the unit hydrograph in c.f.s. per square mile.

VanSickle's work was based on flows from the USGS stream gaging network in and around the city of Houston, Texas. The network covered areas ranging from fully urbanized areas with well-developed drainage systems including storm sewers to undeveloped areas with a natural drainage system. With this range of basic data he was able to show the effect of urban development on the unit hydrograph peak. The relations he developed were presented as curves of t_p and q_p versus a basin factor K for various degrees of urban development as shown in figure 1. The basin factor is similar to the basin parameter used by Eagleson.

$$K = \frac{\bar{L}_t \; \bar{L}}{\bar{S}^{.5}}$$

Where: $\bar{L}_t$ is the total length of drainage channel in the basin in miles and includes all storm sewers 36 inches or larger and all drainage channels large enough to be shown on topographic maps; $\bar{L}$ is the mean basin length in miles; and $\bar{S}$ is the mean basin slope.

Espey took the stream gaging data from Houston and combined it with similar data from Austin as the basis of his research. Using linear regression analysis he derived the following equations for urban areas

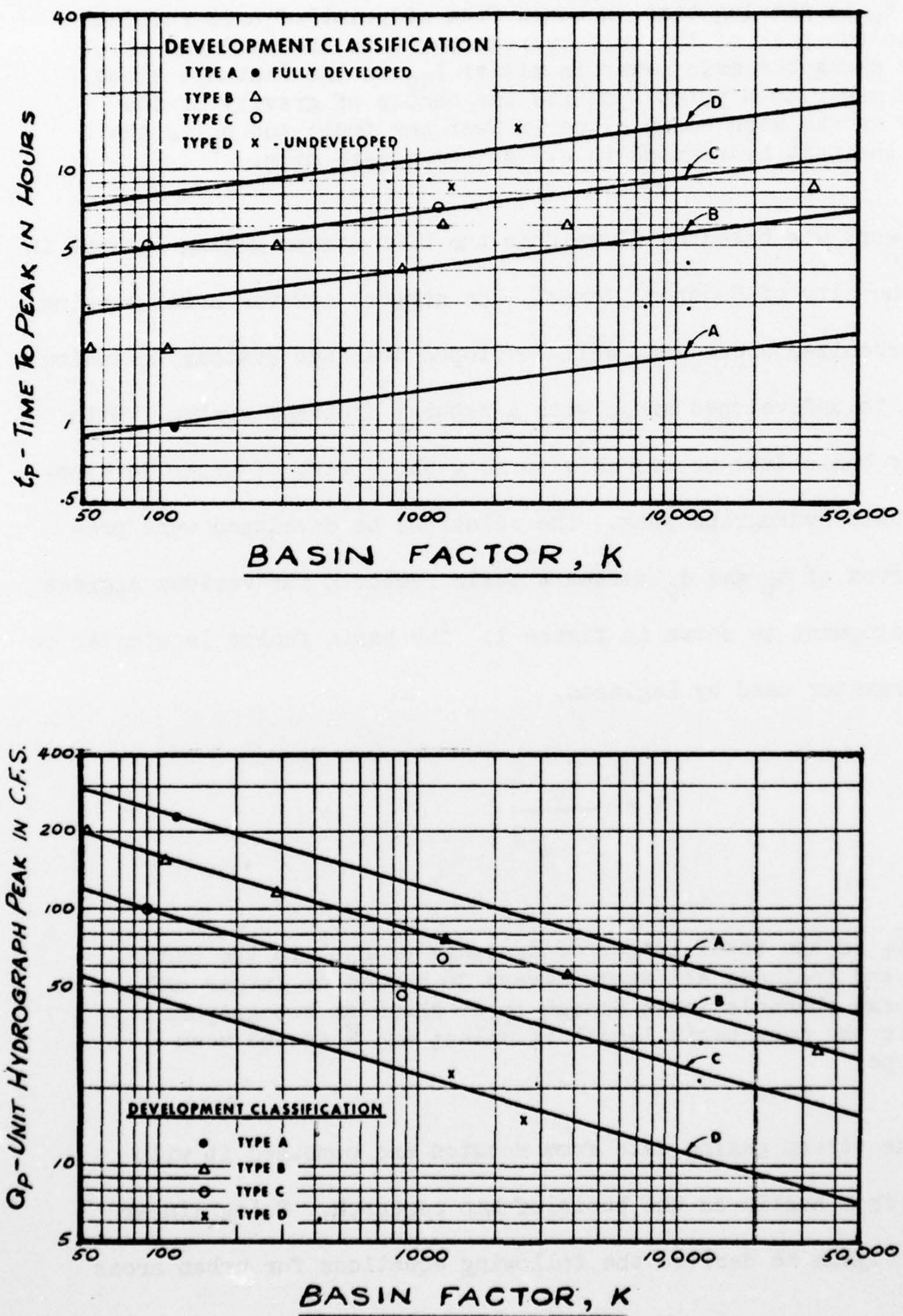
DEVELOPMENT CLASSIFICATION
TYPE A • FULLY DEVELOPED
TYPE B △
TYPE C ○
TYPE D x - UNDEVELOPED
D
C
B
A
t_p - TIME TO PEAK IN HOURS
40
10
5
1
.5
50
100
1000
10,000
50,000
BASIN FACTOR, K
Q_p-UNIT HYDROGRAPH PEAK IN C.F.S.
400
200
100
50
10
5
DEVELOPMENT CLASSIFICATION
TYPE A
TYPE B
TYPE C
TYPE D
BASIN FACTOR, K

FIGURE 1.

relating the time to peak and unit hydrograph peak to the basin characteristics.

$$T_r = 16.4 \; \phi \; L^{.32} \; S^{-.049} \; I^{-.49}$$

$$Q = 3.5 \times 10^4 \; A \; T_r^{-1.10}$$

Where: T_r is the time to peak from the beginning of runoff in minutes; Q is the unit hydrograph peak in c.f.s.; A is the drainage area in square miles; L is the length of the main channel in feet; S is main channel slope in ft per ft; I is the percent of impervious cover; and ϕ is defined as a channel roughness factor ranging from .6 for an extensive channel improvement and storm sewer system to 1.0 for natural channel conditions.

In addition to these three methods formulated specifically for urban areas, the Snyder method and a method developed by Gray (8) may be adaptable to urban areas if sufficient basic data are available. Eagleson and VanSickle derived the Snyder's coefficients for their study areas but found no consistent relation between the coefficients and the degree of urban development.

Gray used a two parameter gamma distribution function, which is analogous to the theoretical expression for the instantaneous unit graph developed by Edson (9) and Nash (10), as a basis for his method. The function is of the form:

$$Q_t/P_R = \frac{25(\gamma')^q}{\Gamma q} \; e^{-\gamma' t/P_R} \left(\frac{t}{P_R}\right)^{q-1}$$

In which: Q_T is the percent of flow from the distribution graph at time t; P_R is the period of rise from the beginning of surface runoff to the peak of the unit graph; γ' is a dimensionless parameter equal to the product of P_R and a scale parameter γ; q is a shape parameter; Γ denotes the gamma function; and e is the base of natural logarithms.

Gray worked with a dimensionless graph reduced from unit hydrographs developed from small basins in the midwest. The parameters q and γ' were evaluated from the dimensionless graphs by curve fitting using a Chi-square test. These parameters were related to each other and to the basin characteristics using regression analysis. As the unit hydrograph developed from this method is a mathematical function it is adaptable for computer use.

To develop a generalized method for determining the unit hydrograph peaks for ungaged urban areas using one of these methods would require a competent staff, time, and sufficient funds to compile the unit hydrographs, basin characteristics, degree of urban development, and the relating functions. In urban areas void of gaging stations, new stations would have to be set up. Although development of a method of this type would involve a large initial investment; this would be offset by the ease of use and no recurring cost.

COMPREHENSIVE SIMULATION MODELS

Anyone who has taken a course in hydrology is familiar with the qualitative or pictorial description of the hydrologic cycle. Over the years numerous measurements of the elements in the hydrologic cycle have been accumulated and numerous research and investigation programs setting forth methods of calculating individual elements have been completed. However, until 1959 no attempt had been made to quantitatively relate all the relationships in the hydrologic cycle. At that time research began at Stanford University leading to the development of the Stanford Watershed Model IV (11). The model is a comprehensive digital simulation model which has been programmed based on mathematical expressions for the values of the elements in the cycle and their relationship to each other. The flowchart of the model shown in Figure 2 may be followed, as rainfall representing inflow is traced through the model.

The watershed is divided into segments with a rain gage in each segment. Rainfall recorded at the gages is first depleted by interception storage, which is an input parameter whose value depends on the type of watershed cover. Rainfall on impervious areas is diverted to channel inflow and the soil cover based on an input parameter relating the effective impervious area to the total impervious area in the watershed.

In the infiltration section of the model, the soil horizon is divided into upper and lower zone storages. The upper zone controls interflow and surface detention leading to overland flow and the lower zone controls

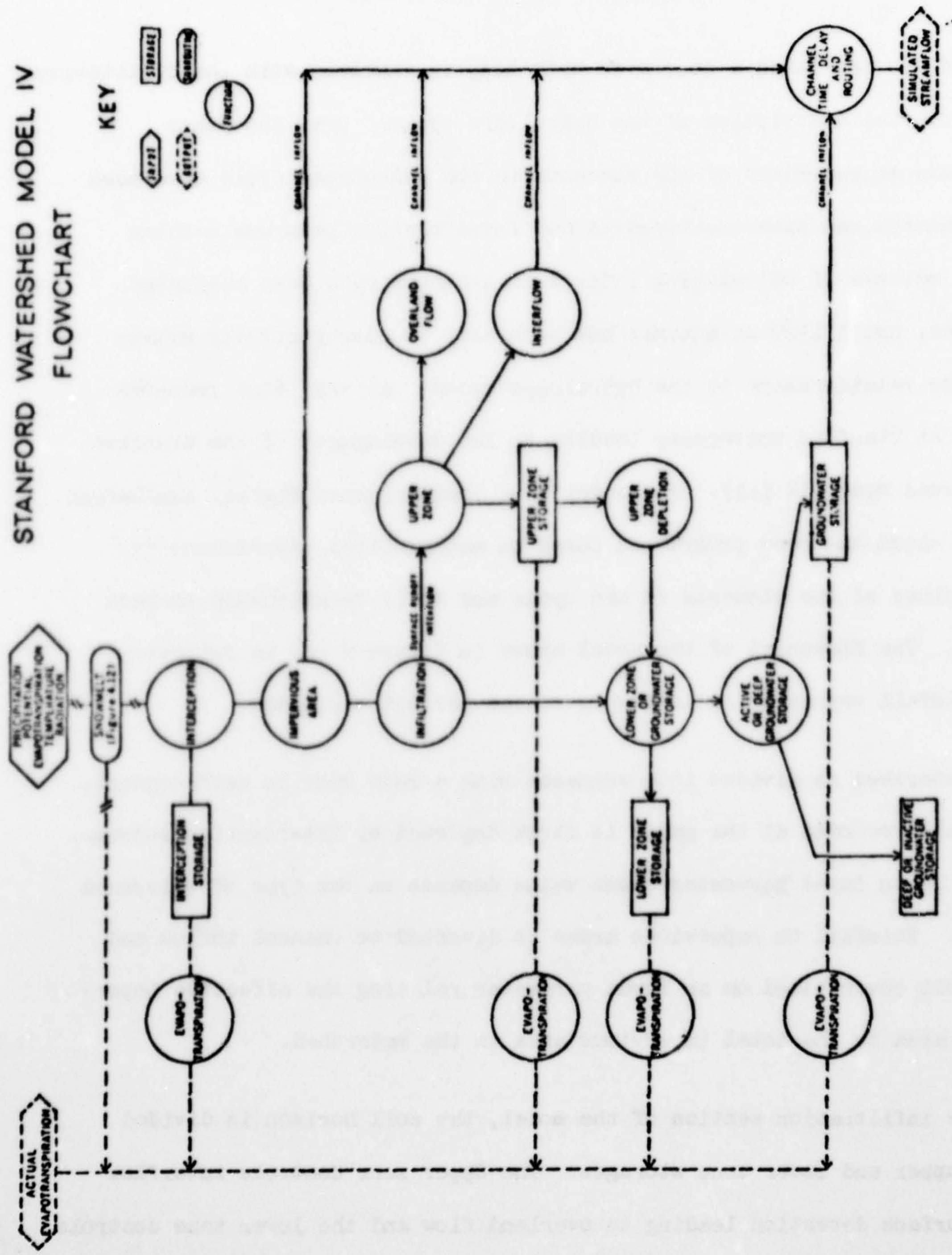
STANFORD WATERSHED MODEL IV
FLOWCHART
KEY
INPUT
OUTPUT
STORAGE
FUNCTION
PRECIPITATION
POTENTIAL EVAPOTRANSPIRATION
TEMPERATURE
RADIATION
SNOWMELT
INTERCEPTION
INTERCEPTION STORAGE
IMPERVIOUS AREA
INFILTRATION
SURFACE RUNOFF
INTERFLOW
UPPER ZONE
UPPER ZONE STORAGE
UPPER ZONE DEPLETION
OVERLAND FLOW
INTERFLOW
CHANNEL INFLOW
LOWER ZONE OR GROUNDWATER STORAGE
LOWER ZONE STORAGE
ACTIVE OR DEEP GROUNDWATER STORAGE
GROUNDWATER STORAGE
DEEP OR INACTIVE GROUNDWATER STORAGE
EVAPO-TRANSPIRATION
ACTUAL EVAPOTRANSPIRATION
CHANNEL TIME DELAY AND ROUTING
SIMULATED STREAMFLOW

FIGURE 2.

the rate of flow to groundwater. A linearized cumulative frequency distribution of infiltration capacity is assumed in the model. The infiltration capacity is divided between flow into the upper and lower zones based on remaining storage in the zones and input parameters controlling net infiltration. The level of interflow relative to overland flow is also controlled by an input parameter. Additions to the overland flow from the upper zone is a function of the actual storage and nominal storage capacity. The percentage of infiltration diverted to the lower zone is a function of the ratio of the actual storage in the lower zone and the storage level at which 50 percent of all incoming moisture moves to groundwater storage.

During this same period evaporation from interception storage, storage in the upper and lower zones, and stream surfaces is taking place. The potential evapotranspiration from the watershed is assumed equal to lake evaporation, which is computed from the input of pan evaporation data. Evapotranspiration occurs from interception storage and the upper zone storage at the potential rate. If the potential is not satisfied in these areas, the remaining potential is satisfied from the lower zone if available. A linear cumulative frequency distribution of evapotranspiration capacity from the lower zone is assumed. Total evaporation from the zone is dependent on an input parameter and the storage remaining. Evapotranspiration from stream surfaces and groundwater are based on the potential rate and input parameters indicating their areas.

The outflow hydrograph from overland flow is based on the Chezy-Manning equation and an empirical relation between detention storage and outflow depth while the interflow hydrograph is based on the interflow detention storage and an input depletion parameter. The groundwater outflow is computed from the saturated groundwater zone at the stream channel and the groundwater energy gradient. The channel inflow hydrograph is the sum of overland, interflow, and groundwater flow hydrographs. During nonrainfall periods, the groundwater flow is controlled by a recession constant which produces the logarithmic depletion curve. Deep percolation to inactive groundwater is assigned by an input parameter.

The Clark method (12) which utilizes a time-area curve and reservoir routing was modified to reflect only channel attenuation as the land system has already been included in the channel inflow hydrograph. The time-area curve was redefined as a channel-delay histogram which represents the time for the contributing areas to flow to the outlet of the watershed. The travel times are based on uniform flow velocities for a flow of once in 10 or 20 years. The histograms from all the segments in the watershed are added together at the outlet and then routed by a reservoir routing to produce the outflow hydrograph.

This brief description of the model is provided not to present details of the calculations represented in the model but to emphasize the number of variable input parameters that must be selected. Before simulating a period of flows from a watershed, the values of the parameters are

varied until the model accurately reproduces an observed set of hydrographs from the basin. This testing of the input parameters would seem to eliminate the model's use in computing peak discharges from ungaged urban areas. However, further simulation of urban watersheds for which there are flow records for verification of the parameters is planned. With sensitivity studies of the parameters, their values can be accurately related to the physical characteristics of the basin; thus permitting simulation of peak flows from urban areas with an acceptable degree of confidence without verification.

The cost of developing a comprehensive simulation model would appear to be out of reality as 4 years have been required to bring the Stanford Model to the present state of development. An IBM 360 Model 65 computer with 430k bytes of memory and 4 disk storage units is required for the Stanford Model and the Districts' computers do not have this capability. This would require that the problem would have to be taken to the model, which can be costly and inconvenient. The major advantage of the simulation model is that long periods of complete flow records can be generated along with the peak discharges.

RECOMMENDATION

Development cost, ease of use, and general applicability are the three most important characteristics that should be considered in selecting one of the methods for use in computing peak discharges from ungaged urban areas. Physical models can be eliminated from consideration because of their high cost, limited applicability, and the special facilities required. The use of comprehensive digital simulation models in the immediate future

will probably be limited to research in the major universities because of the computer capability required. The simulation model may prove economical where a long period of record is required for design of an urban flood control project in an extremely high-damage potential area. It is hard to visualize the use of the model in a flood plain information study with a total budget of $25,000.

Synthetic unit hydrographs appear to be the method most adaptable for use in Corps studies, even though the cost of developing the method will be high. If a sampling of basic data can be provided from urban centers over the country, the resulting method will probably have general applicability. Application of the method to compute peak discharges will be very similar to the Snyder method for rural areas. The working-level hydraulic engineer, being familiar with the Snyder method, will have no trouble in coming up with timely solutions using the new method.

It is hoped that one recommendation passed on to OCE from this seminar is the need for a unified method of computing peak discharges from ungaged urban areas and that development of a synthetic unit hydrograph method appears the most feasible solution. The method, along with other recommendations from this seminar, should be included in a future EM on urban hydrology.

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PROCEEDINGS OF A SEMINAR ON URBAN HYDROLOGY HELD ON 1-3 SEPTEMB--ETC(U)
SEP 70 E F CHILDS, G S HARE, D L ROBEY

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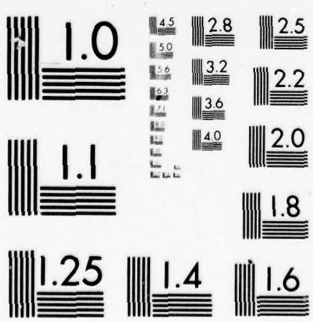

MICROCOPY RESOLUTION TEST CHART
NATIONAL BUREAU OF STANDARDS-1963-A

A UNIFIED METHOD FOR COMPUTING PEAK DISCHARGE
FROM UNGAGED URBAN AREAS FOR
CORPS OF ENGINEERS STUDIES

Discussion

Question, Mr. Beard: Considering that the Corps of Engineers activities in urban hydrology in the next 10 to 20 years might be measured in hundreds of millions or even billions of dollars, do you feel that ease of use, difficulty of using large computers and cost of development should be given large weight in developing or selecting a unified method of runoff computation? Also, don't you think that a comprehensive simulation model could be developed that would be essentially as simple as the unit hydrograph methods?

Reply, Mr. Henson: It was not the intent of the paper to eliminate Corps development of a CSM (comprehensive simulation model). Although not stated in the Recommendations, but in the Introduction, the emphasis is on the current need of a unified method. I believe a unified method could be developed in a short time using available urban gaging records and the synthetic unit hydrograph approach. Thus, it would be in use before a CSM could be developed. In my opinion development of a CSM should be included as a long range program, say within the next 10 years. Also in the long range program the synthetic unit hydrograph method could be updated and improved. The result would be a CSM and a synthetic unit hydrograph approach which would complement each other as neither is the most efficient answer to all peak flow problems.

I have no basis for an opinion as to whether a CSM could be developed which would be as simple to use as the unit hydrograph.

Comment, Mr. Northrop: It would add to this paper to give a brief summary of work being done by other universities with the hydrologic cycle models. I could furnish information on the Kansas work. Perhaps Mr. Beard could fill you in on work in other areas. I would like to see these models adopted to urban hydrology on a long range basis.

Reply, Mr. Henson: It was pointed out by several participants in the seminar that comprehensive simulation models were also being developed at Kansas University, Kentucky University, and by Dawdy with the USGS. However, to include a summary of these models is not possible in the time allowed for response. The general consensus of the participants was that further research, studies, and development of a comprehensive simulation model for Corps use should be initiated now with ultimate development expected within the next ten years.

TULSA DISTRICT METHOD OF URBAN HYDROLOGY

by S. E. Jones[1]

1. Introduction. In earlier years the design of flood control projects in the Tulsa District, while directed toward the protection of urban areas, was actually a function of runoff from undeveloped areas in the upper parts of the watersheds. In very few instances was the effect of runoff from the urban areas themselves a major factor in the design of the flood control projects. As a result, the vast majority of the effort in hydrologic studies for such planning activities has been directed toward the determination of storm runoff rates from undeveloped areas and the routing of these floods through the various reservoirs or channel sections to be improved.

In recent years the increase in urban development has been tremendous. This urban growth replaces forests and fields with the paved areas and structures of residential, commercial, and industrial development. Under these circumstances the design of flood control projects must of necessity take into account the effects of this urban development on storm runoff rates. This is particularly true at the present time when Flood Plain Information and Local Protection Studies are requested for small drainage areas which are at the present time, or will be in the foreseeable future, largely overbuilt by urban development.

2. General Effects of Urban Development on Storm Runoff. The effects of the urban development of a watershed are shown in two ways:

(1) Reduction in infiltration losses by covering the permeable soils with impermeable streets, parking lots, and roofs, and

(2) Provision of more hydraulically efficient channels through which

[1]Hydraulics Branch, Tulsa District

the storm runoff can flow.

This results in an overall increase in storm runoff because of the reduced infiltration losses and in increases in peak runoff because of shorter concentration time in the more efficient drainage system. While it is not difficult to see that the general effect of urban development would be to increase both total runoff and peak runoff rates, it is extremely difficult to develop relationships which accurately define the extent of these changes in runoff rates and characteristics (1).

3. Tulsa District Method of Computation.

a. General. In the Tulsa District, two types of problems exist which require studies to determine effects of urbanization on runoff characteristics. The first type of study is the Flood Plain Information Report. These reports contain information showing flood hazards under existing conditions. The second type is the development of design discharges for local protection projects to be built under Section 205 of the 1948 Flood Control Act as amended by Public Law 87-874.

The purpose of this paper is to present the method used by the Tulsa District Corps of Engineers to determine the effects urbanization has to small areas, with respect to surface runoff. So far the method presented in this report has been applied only to synthetic data. Since there are no small drainage areas (drainage areas less than 10 square miles, usually the case with urban areas) with stream flow data available to pattern a study against, no concrete conclusions as to the accuracy of this method have been made.

The most commonly used procedure for developing a hydrograph of discharge

at a selected location utilizes the unit hydrograph method. The unit hydrograph method is universally accepted and has been used by the Tulsa District for many years. Where recorded hydrographs are not available to permit derivation of unit hydrographs (which is normally the case where small drainage areas are considered), the synthetic unit hydrograph as presented by Franklin F. Snyder in EM 1110-2-1405, Flood-Hydrograph Analyses and Computations is usually selected. In the unit hydrograph derivation, the coefficients C_t and C_p depend on drainage basin characteristics. Since urban development does affect these basin characteristics, an adjustment applied to C_t and C_p will in effect account for urbanization to the unit hydrograph. For this reason, we have selected the method by Franklin F. Snyder to adjust the unit hydrograph for built-over conditions. This method was published in an article titled "Synthetic Flood Frequency" in the October, 1958, issue of the American Society of Civil Engineers' Journal of the Hydraulics Division.

Mr. Snyder based the development of his method on a study of drainage areas in the vicinity of Washington, D. C. He found that after a value of C_t for natural basins has been selected for a particular region, it is considered that the percentage of open drainage channels which have been eliminated and the percentage of the basin which has been storm-sewered are the principal factors to use for interpolating between the selected value of C_t for the natural basin and the value of 0.42 for the complete storm drainage condition. Mr. Snyder states that data were not available to define the relationship exactly, but those which were available indicated that reasonable results are obtained by averaging the two percentage

factors and interpolating on a straight line between the natural C_t and 0.42. The equation is:

$$\text{Adj. } C_t = \text{Nat. } C_t - \left(\frac{\%\text{ Storm-sewered} + \%\text{ Nat. channel eliminated}}{2}\right)(\text{Nat. } C_t - 0.42)$$

The 0.42 is considered to be the C_t for a fully storm-sewered condition.

b. <u>C_t Determination</u>. In the Tulsa District, we have many streams without stream flow gages. Since many of our studies involve streams without gages, it became essential that some method of selecting unit hydrograph coefficients for ungaged streams be developed. In order to establish any reliability on the method selected, areas with gaged flows were investigated. The late Mr. Benjamin G. Baker, a hydraulic engineer in the Tulsa District, established a curve of weighted stream slope versus C_t. Mr. Baker, who also worked at other districts while employed with the Corps of Engineers, collected data from many streams in these districts where unit hydrographs were derived from known discharges and storm runoff. A plot of this data was made on logarithmic paper and a best fit curve was constructed. This curve is shown on Figure 1. The Tulsa District uses this curve as a guide in selecting C_t values in areas which do not have gaged flows. If a gaged stream with similar basin characteristics is located within the vicinity of the study area, the C_t value for the gaged stream is used in conjunction with Figure 1 in selecting a C_t for the ungaged stream, generally by going parallel to the guide curve.

c. <u>C_p Determination</u>. It is also necessary to determine C_p, or develop some substitute for obtaining the results C_p gives. Since C_p is a function of q_p and t_p, we cannot form any correlation since both q_p and t_p are unknowns. However, if we are working with unit hydrographs derived from gaged data, the q_p can be computed directly using the peak rate of discharge

of the unit hydrograph and the drainage area ($q_p = \frac{Q_p}{A}$). With C_t established and using $(LLca)^{0.3}$ where L and Lca can be measured from topographic maps, the t_p can be computed $t_p = (C_t)(LLca)^{0.3}$. A logarithmic plot was made by plotting t_p versus q_p for all the drainage areas with defined unit hydrographs and a best-fit curve was constructed as shown on Figure 2. This curve is used by the Tulsa District as a guide in the selection of q_p for the ungaged drainage areas in the same way Figure 1 of C_t versus weighted slope was used.

d. Unit hydrograph. With a topographic map to determine the drainage area and weighted stream slope and Figures 1 and 2 to select C_t and q_p values, we can now produce a synthetic unit hydrograph for natural conditions. To adjust the natural unit hydrograph to account for urban development, the percent of the drainage area which has storm sewers and the percent of open drainage channels eliminated by the storm sewers must be known.

4. Flood Plain Information Reports. In Flood Plain Information reports, we are interested in determining urban development for present conditions. As a general rule, we consider built-over areas to be fully storm-sewered. The method used to determine present built-over conditions is to examine a recent aerial photograph of the study area. If aerial photographs are not available for the area, topographic maps can be used successfully when coordinated with a field reconnaissance. The percent of natural channel eliminated by storm sewers can be obtained from a field reconnaissance or from the local government agency involved. With this information, the equation shown on page 4 can be used to determine the adjusted C_t for a built-over area.

5. Local Protection Studies. For local protection projects, we are

interested in both current and future urban development. For these projects, we obtain an evaluation of the growth potential and a projection of the expected percent of urban development in the proposed project life. It is generally assumed that the percent of natural channel eliminated is the same as the percent of development, unless there are relatively large natural drains in the area which could be used as collectors for smaller storm sewers. The equation shown on page 4 is again used to determine the C_t adjusted for a built-over area. Using this C_t, the unit hydrograph for the urban area is then developed.

6. Example. The study area is located on the boundary of the urban limits of a city. The drainage basin characteristics of the area as determined from a topographic map are:

$$D.A. = 2.44 \text{ sq. mi.}$$

$$L = 3.0 \text{ mi.}$$

$$Lca = 1.2 \text{ mi.}$$

$$(LLca)^{0.3} = 1.47$$

$$S_{st} = 0.003 \text{ ft./ft.}$$

Since there are no gaged flows in the vicinity, we use Figure 1 to find that $C_t = 1.18$ for natural conditions. From this it is determined that $t_p = C_t(LLca)^{0.3} = 1.73$. Using Figure 2 with $t_p = 1.73$, we read $q_p = 235$ and from this, $640\ C_p = (q_p)(t_p) = 407$. The following calculations are then made:

$$t_r = t_p/5.5 = 0.31$$

$$\text{Use } t_R = 0.25$$

$$t_{PR} = t_p + 1/4(t_R - t_r) = 1.71$$

$$q_{PR} = 640\ C_p/t_{PR} = 238$$

$$Q_{PR} = q_{PR}(D.A.) = 581$$

It is found that the peak rate of discharge of the unit hydrograph is 581 c.f.s. for natural conditions.

Based on an evaluation of the growth potential, this area is expected to be 75 percent built-over within the design life of the proposed project. It is also expected that 50 percent of the natural channel will be eliminated by storm sewers based on a field reconnaissance of the area. Using Snyder's method to adjust for built-over conditions,

$$\text{Adj. } C_t = \text{Nat. } C_t - \left(\frac{\%\text{ built-over} + \%\text{ Nat. channel elim.}}{2}\right)(\text{Nat. } C_t - 0.42)$$

$$\text{Adj. } C_t = 1.18 - \left(\frac{.75 + .50}{2}\right)(1.18 - 0.42) = 0.70$$

we find that the adjusted $C_t = 0.70$. After the following calculations are made,

$$t_p = C_t\,(LLca)^{0.3} = 0.70\,(1.47) = 1.03$$

$$t_r = t_p/5.5 = 0.19$$

$$\text{Use } t_R = 0.25$$

$$t_{PR} = t_p + 1/4(t_R - t_r) = 1.03 + 1/4(0.25 - 0.19) = 1.05$$

$$q_{PR} = \frac{640\ C_p}{t_{PR}} = 407/1.05 = 388$$

$$Q_{PR} = q_{PR}(D.A.) = (388)(2.44) = 947$$

it is found that the peak rate of discharge of the unit hydrograph for future built-over conditions is 947 c.f.s. This is an increase of 63.0 percent over the natural conditions.

Had there been a gaged stream in the vicinity of the study area with similar basin characteristics, it would have been used in the following manner:

We will assume the gaged stream has a weighted slope = 0.0011, C_t = 1.26, t_p = 11.0, and q_p = 29. To obtain the natural C_t for the study area, the weighted slope and C_t for the gaged stream are plotted on Figure 1. A line is projected through this point, parallel to the curve on Figure 1, to the weighted slope of the study area. The natural C_t is then read from the intersection of this line and the study area slope. In this case the C_t would be 0.83. The new t_p is calculated to be $t_p = C_t(LLca)^{0.3}$ = (0.83)(1.47) = 1.22.

The t_p and q_p of the gaged stream are now plotted on Figure 2 and a line projected through this point in the same manner as on Figure 1. The q_p for the study area is then read from the intersection of this line and a t_p of 1.22. This gives a natural q_p of 200 for the study area. The remaining calculations are the same as those presented in the previous example.

7. Runoff. Initial losses and infiltration indices have been computed for several major storms on gaged flows within the Tulsa District. Although the analysis indicates a wide range of infiltration rates under apparently similar antecedent moisture conditions, the studies are used as a guide in selecting loss rates for natural conditions on projects located in the vicinity of the gaged flows. In areas where no infiltration studies have been made, we base the loss rates on areas with similar characteristics where previous projects have been studied.

The Tulsa District has no set criteria for adjusting rainfall losses for built-over areas. Each project engineer uses his judgment in adjusting

the natural loss rates to account for urban development. Previous urban studies, and the type of development expected such as residential or commercial, are used by the engineer to help select loss rates for urban conditions. A curve of % impervious area vs % runoff would be helpful if adequate data were available to develop it.

8. Problems. Although urban hydrology is not new to the Tulsa District, problems caused by the encroachment of urban development on natural surroundings have become much more critical in recent years. The major problem is the lack of information required to make accurate land use studies and data to determine the effects of urbanization on the hydrologic characteristics. The hydrologic data collection has been primarily associated with the development of large flood control projects and navigation systems. As a result, very little data are available for small drainage areas. The curves shown on Figures 1 and 2 are based on information obtained from large drainage areas and may not be reliable for the small drainage areas usually encountered in urban hydrology.

9. Summary. The method developed by Snyder was based on a study of the Washington, D. C. area. The results of his study, particularly the C_t of 0.42 for a fully developed area, may not be completely applicable to the Tulsa District. However, runoff from built-over areas should have some characteristics common to most urban areas and we feel that his method is the best that we have at the present time to adjust for urban development.

Future studies will establish a method of determining the effects urban development has on storm runoff in the Tulsa District. The Tulsa District has very limited data on small drainage areas, both urban and rural. A

research will be made to determine what information will be available from local, stage and federal agencies in the future.

The Tulsa District is honored to participate in this seminar. We feel that the information obtained from the Divisions and Districts represented here will be beneficial in helping us improve our method of analyzing the effects of urbanization.

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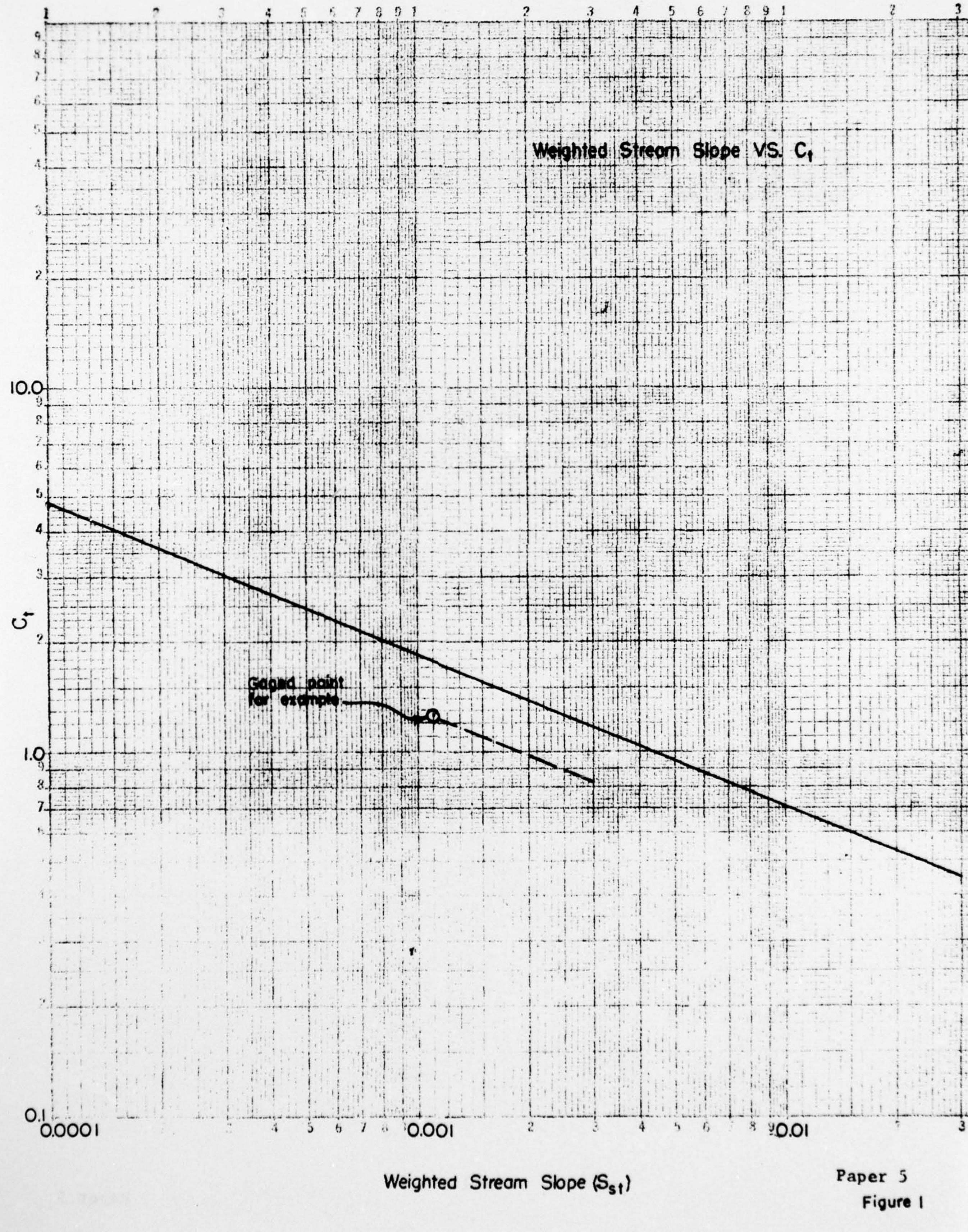
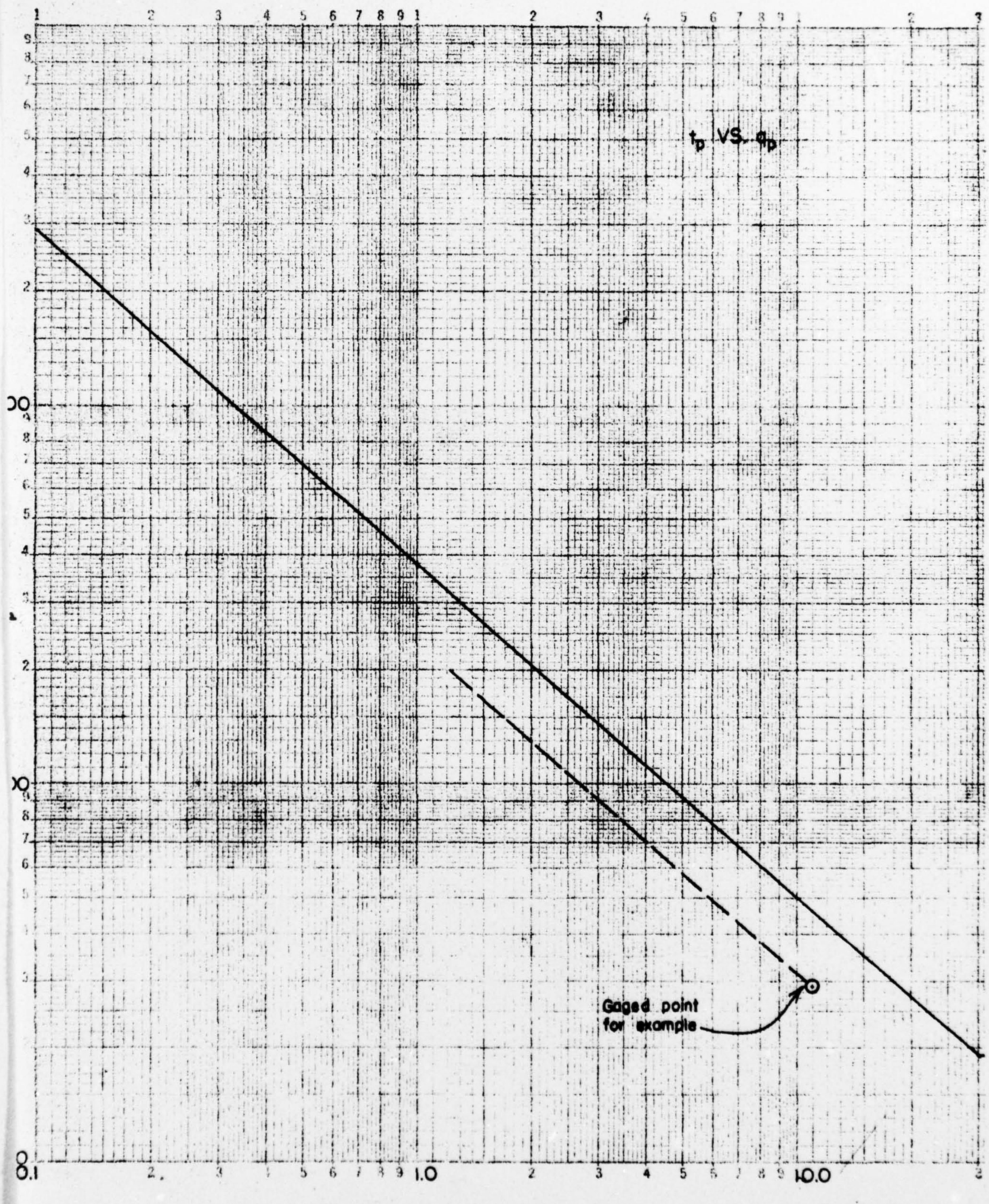


Paper 5
Figure I



Paper 5

Figure 2

TULSA DISTRICT METHOD OF URBAN HYDROLOGY

Discussion

Question, Mr. Hare: In Snyder's Washington, D. C. studies, is it correct to assume that $C_t = 0.42$ is applicable to all areas without respect to weighted stream slope or size of drainage area?

Reply, Mr. Jones: I feel that this could give answers that are not correct because, in my opinion, the drainage basin characteristics do have an effect on C_t whether the area is rural or urban. However, at the present time we have no data to check the C_t value of 0.42 for fully urbanized areas.

Comment, Mr. Henson: The October 1958 paper by Snyder does not present a relation to adjust 640 C_p for increased urbanization. Therefore, in the method you have presented a constant value of 640 C_p was used for present and fully urbanized conditions. Van Sickle found in his analysis of records on Brays Bayou that the value of 640 Cp also decreased with urbanization. However, C_t decreased faster than 640 C_p which results in an increase in the peak of the unit hydrograph for urban conditions.

Reply, Mr. Jones: C_p will probably change also with urbanization, but Snyder chose only to use the C_t constant to change the unit hydrograph for urban conditions. It may be that his change in C_t could give the same results as changes made in C_t and C_p combined.

Question, Mr. K. Johnson: Are generalized curves developed for unit hydrographs with comparable unit durations? If not, wouldn't use of these data for small areas (such as that used in example) produce questionable results?

Reply, Mr. Jones: No, The use of the curves developed for the large drainage areas would produce questionable results in applying them to the determination of C_t and C_p values for small drainage areas. The results should be checked against gaged data.

Question, Mr. Childs: The analysis using the unit hydrograph to determine present and future flows results in peak discharges for unrestricted runoff. Has the effect of storage been considered in modifying these flows and used in developing the stage-frequency relationships for the economic analysis?

Reply, Mr. Jones: The peak discharges were determined by unit hydrograph analysis at the point where the tributaries entered the main flow channel. The flows were then collected and routed down the creek under study. This routing considers storage in the overbank in determining peak discharges at various predetermined locations as the peak moves downstream.

Comment, Mr. Beard: It is interesting to note that there is a move on in the Chicago area that might spread to other areas to require storage to offset the effects of urbanization in such a way as not to increase flows at specified points downstream. While this can be extremely difficult to administer from a technical viewpoint, perhaps the basic idea is sound.

Reply, Mr. Jones: This might be a good idea if ponding areas for storage can be found which won't upset local property owners. If water stands in their lawn or driveway frequently, they will be asking for improvements to move the water.

Question, Mr. Beard: How does the Tulsa District determine rainfall for specified exceedence frequencies and do you assume that runoff frequency corresponds to the rainfall frequency?

Reply, Mr. Jones: The Tulsa District uses technical paper 40 for rainfall frequency and assumes that runoff frequency corresponds to the rainfall frequency.

Question, Mr. Northrop: Is there any provision to check time of peak against field data obtained during the preparation of the study? This would help to verify the results obtained by this version of the Snyder Method.

Reply, Mr. Jones: This is a good suggestion and is certainly a good policy to stablize your studies. Also, using a largely unproven procedure would be risky unless double checked by other methods such as Clark's or the Rational Formula as well as field observations.

SYNTHETIC UNIT HYDROGRAPH RELATIONSHIPS
TRINITY RIVER TRIBUTARIES
FORT WORTH-DALLAS URBAN AREA

by Thomas L. Nelson[1]

The Fort Worth-Dallas area is a rapidly expanding urban area. An excellent freeway system and a highly developed secondary road system in the Dallas-Tarrant County area are largely responsible for the rapid conversion of pasture and cropland into residential and industrial developments. The suburban cities of Garland, Richardson, Mesquite, Grand Prairie, Arlington, Benbrook, and North Richland Hills are among the fastest growing cities of the United States, several of them increasing in population from a few thousand in 1950 to 30,000-40,000 in 1960 and doubling in population during the decade of the 1960's. Although much of the growth has been orderly and well planned, development in many flood-prone areas has resulted in severe flood damages and in the necessity for the construction of local or Federal flood control works. Four multiple purpose reservoirs have been constructed by the Corps of Engineers in this general area and a fifth is in advanced planning stages. Two major leveed floodways have been constructed by the Corps through the cities of Fort Worth and Dallas, and the West Fork Floodway and an extension of the Dallas Floodway are in the advanced planning stages. A channel improvement and levee project has been constructed along Big Fossil Creek in Richland Hills and advanced planning has begun on the Duck Creek channel improvement in Garland.

To reduce future need for additional flood control structures, it has become imperative that guidance be provided in directing

[1]Hydrology Section, Forth Worth District

future development. The Federal flood plain information and flood insurance programs are initial steps in providing this guidance.

To date, the Fort Worth District Corps of Engineers has prepared 14 flood plain information reports, has 6 others in various stages of completion, and has a backlog of at least 19 additional applications at the present time. Eighteen of these 39 reports are in the Fort Worth-Dallas area and 16 of the 18 are along ungaged tributary streams.

The demand for this large number of studies has required the development of a procedure which would provide a reasonably accurate method of determining present and projected discharges on small ungaged streams. Since these streams are located in urban areas with watersheds in varying degrees of development, a method of accounting for the present and anticipated future degree of urbanization was also needed.

An ideal method for determining the effect of urbanization on streamflow is to gage the streamflow over the long period during which the watershed changes take place and to correlate the changes with the variations in streamflow characteristics. Lacking such long-term record, one might study several watersheds which were hydrologically similar but which had experienced different degrees of urbanization. The latter method is the alternative which the Fort Worth District found available to it and with which this paper will deal.

For this analysis, records were available from eight different

streamflow gages along small streams in the Fort Worth-Dallas area with drainage areas varying from 7.5 to 130 square miles (figure 1). Records from five of the eight gages are part of the special urban hydrology studies currently being made by the U. S. Geological Survey in cooperation with local interests in the four Texas metropolitan areas of Dallas, Houston, San Antonio, and Austin.

The U.S.G.S., in cooperation with the City of Dallas, began its program of basic data collection in that city in 1961. The purpose of the program was to allow the evaluation of the hydrologic factors affecting floods in several small streams in the north part of the city. Since 1962, the U.S.G.S. has prepared six reports on floods occurring in this area. The Survey has also published annual reports presenting detailed basic hydrologic data for these watersheds.

The watersheds studied have experienced various degrees of development. For example, the upper White Rock Creek watershed above the upper gage at Keller Springs Road has experienced almost no development. The adjacent watershed of Turtle Creek, on the other hand, is completely urbanized. Not only is the watershed of Turtle Creek urbanized, but it contains a number of low-head channel dams which produce an almost continuous chain of pools along much of the natural channel which rapidly transmit flood waves downstream.

Other streams in the north part of Dallas which were included in the U.S.G.S. programs are Bachman Branch and Joe's Creek. These latter streams are tributary to the Elm Fork Trinity River, while White Rock and Turtle Creeks are tributary to the main stem of the

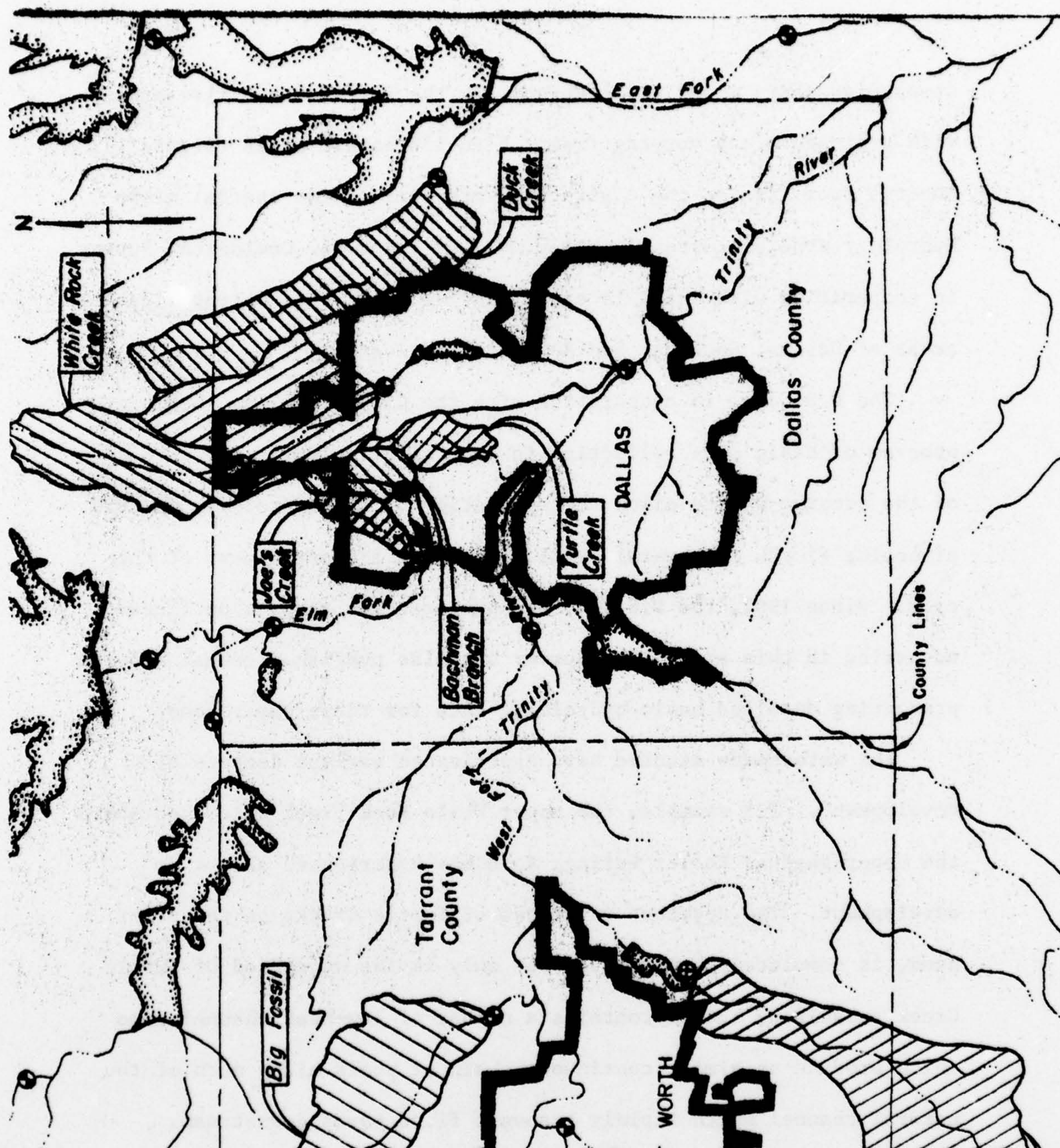

N
White Rock Creek
Duck Creek
East Fork
Trinity River
Dallas County
DALLAS
Turtle Creek
Joe's Creek
Bachman Branch
Elm Fork
Trinity
County Lines
Tarrant County
West Fork
Big Fossil
FORT WORTH
Clear Fork
Village Creek

FIGURE 1

Trinity River. Three other streams which were included in the current study are Duck Creek which flows through Garland, a Dallas suburb, into the East Fork Trinity River; Big Fossil Creek, which flows through Richland Hills, a Fort Worth suburb; and Village Creek which flows near the eastern limits of Fort Worth. The latter two streams are tributary to the West Fork Trinity River.

As previously stated, to allow the type of analysis which we are making, the streams studied should have similar hydrologic and hydraulic characteristics. Except for the differences in the degree of development, these streams are similar. The following U.S.G.S. description of the north Dallas streams (1) applies equally well to the other streams included in the study: "Within the reaches subject to extreme flooding, the stream channels are similar in shape and capacity . . . . Each channel generally consists of a wide flood plain along both banks in the upper reaches and a relatively narrow flood plain in the lower reaches. As tributaries approach the (main stream), the flood plain of the (main stream) provides a wide area of overflow. Much of the flood plain is in urban areas and is covered with well-kept lawns and paved streets. The rest of the flood plain is mostly covered with natural grasses on abandoned farmland. The main channels of the major tributaries, which are generally incised to limestone bedrock are approximate trapezoids cut into well-compacted alluvial material. Trees and brush growth along both banks of the main channels are fairly heavy. The low-flow channels are in solid limestone and

contain some sand and gravel riffles. Little scour --- (occurs) --- except at bridges and constrictions where (during large floods) a large part of the water --- (flows) --- over the roads."

A list of the streams and pertinent data for the streamflow gages for which data were analyzed for this study are presented in table 1.

Unit hydrograph determinations were made for each of the eight stream gages for each storm period which could be analyzed. The methods utilized in the analysis were as set forth in the Corps' EM 1110-2-1405, "Flood Hydrograph Analyses and Computations." The resulting Snyder's coefficients q_p and t_p, where

q_p = unit hydrograph peak in cfs per square mile

t_p = lag from midpoint of unit duration to the time of unit hydrograph peak in hours

were combined for each station and the average values determined. The averages are given in table 2.

In order to generalize the results of these studies for use in ungaged areas, the coefficients needed to be correlated with some measurable watershed characteristics. A review of available literature indicated that the t_p generally correlated well with the parameter $LL_{ca}/S^{0.5}$ where

L is the length of the longest water course in miles,

L_{ca} is the distance in miles along the main water course from the station to the centroid of drainage area,

S is the weighted main stream slope as defined in EM 1110-2-1405 in feet per mile.

TABLE 1

PERTINENT DATA - STREAMFLOW GAGES

Watershed Number	Stream	Station	Drainage area sq mi	Period of Record	Location
1	White Rock Creek	Keller Springs Road	29.4	Aug 1961 to date	North Dallas
2	White Rock Creek	Greenville Avenue	66.4	Aug 1961 to date	North Dallas
3	Turtle Creek	Dallas	8.0	Apr 1948 to date	North Dallas
4	Bachman Branch	Midway Road	10.0	Oct 1963 to date	North Dallas
5	Joe's Creek	State Highway 114*	7.5	Oct 1963 to date	North Dallas
6	Duck Creek	Garland	31.6	Jan 1958 to date	Garland
7	Big Fossil Creek	Haltom City	53.0	Jan 1959 to date	Haltom City
8	Village Creek	Handley	130.0	1925 to 1930	East Fort Worth

*Partial record station

TABLE 2

AVERAGE SNYDER'S COEFFICIENTS
ADJUSTED 1-HOUR UNIT HYDROGRAPHS

Watershed Number	Stream	Gage	q_p (cfs/sq mi)	t_p (hours)
1	White Rock Creek	Keller Springs Road	196	3.33
2	White Rock Creek	Greenville Avenue	160	4.10
3	Turtle Creek	Dallas	600	0.75
4	Bachman Branch	Midway Road	480	0.85
5	Joe's Creek	State Highway 114	250	1.25
6	Duck Creek	Garland	150	2.64
7	Big Fossil Creek	Haltom City	136	4.00
8	Village Creek	Handley	89	5.20

The pertinent watershed constants were determined for this study and are listed in table 3.

The number of data points available from this study (8) are not sufficient to establish the slope which the relationship between t_p and $LL_{ca}/S^{0.5}$ should have. The referenced review of the literature, however, indicates that the slope has been well established in California and has been substantiated at least in Louisville and in Houston. The Corps of Engineers used a similar analysis in defining the lag for streams in southern California. The procedure, presented by Linsley (3) accounted for the scatter of data with separate parallel curves for mountain areas, foothill areas, and valley areas. The lag in the California analysis was defined as the time from beginning of rain to the centroid of runoff (figure 2). Eagleson in a later analysis (4) presented data for sewered urban areas in Louisville, in which t_p was defined according to Snyder's definition (figure 2). Although Eagleson presented few data points, he concluded that lines were indicated having the same slope as the natural areas of the southern California relationship. Also, the urban area data "continued the trend toward shorter lag time for a given value of $LL_{ca}/S^{0.5}$, due presumably to the higher velocities caused by more uniform surface character."

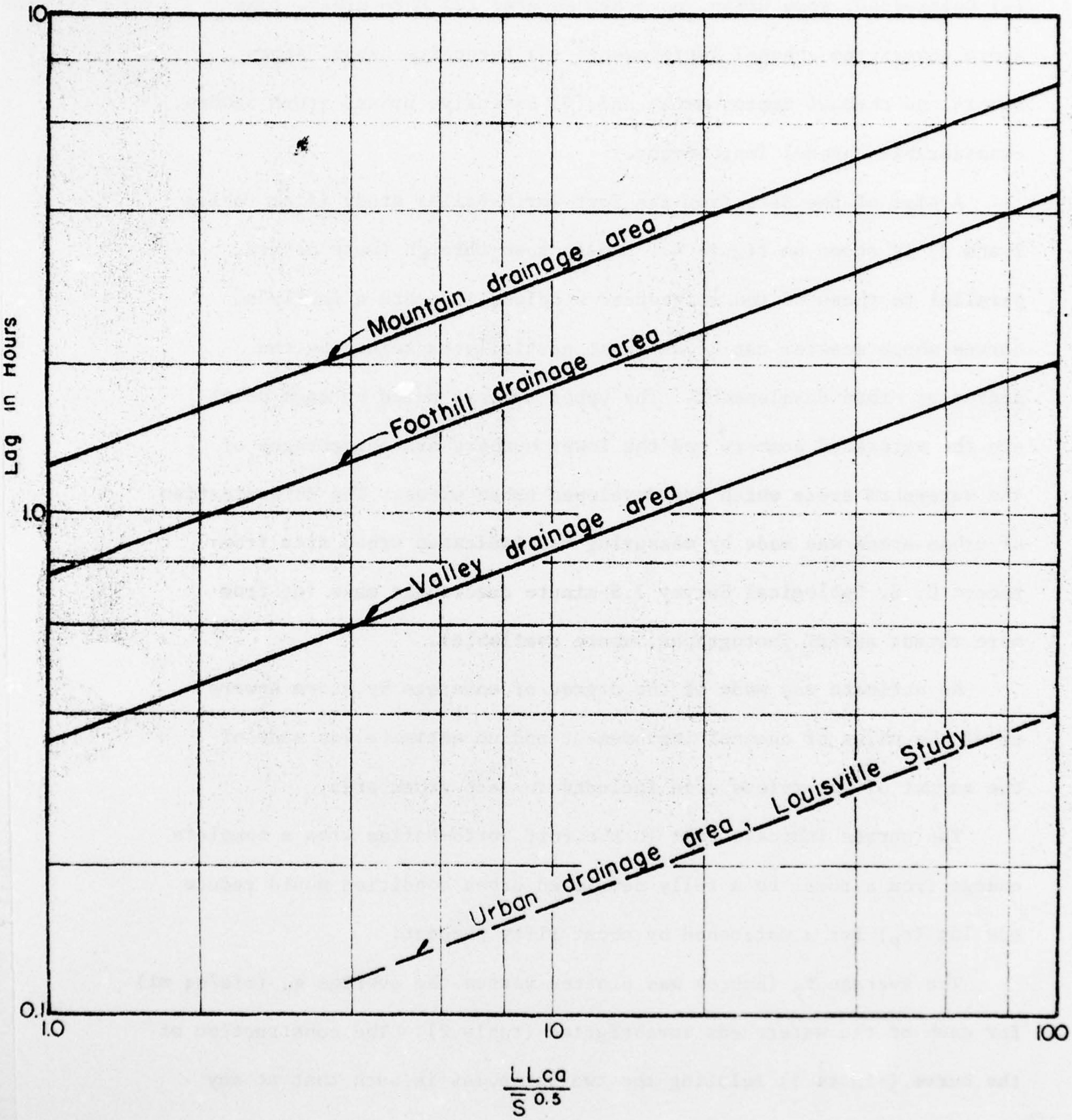
More recently, VanSickle (5) presented results of a study for drainage design in Houston wherein he developed a similar correlation, maintaining the same slope as the original California study. VanSickle explained the scatter of data points by subdividing the

TABLE 3

WATERSHED CONSTANTS

Watershed Number	Stream	Gage	L miles	L_{ca} miles	S ft/mi	$S^{0.5}$	$LL_{ca}/S^{0.5}$
1	White Rock Creek	Keller Springs Road	14.5	7.3	14.63	3.83	27.6
2	White Rock Creek	Greenville Avenue	23.0	11.7	11.77	3.43	78.4
3	Turtle Creek	Dallas	5.7	2.7	27.35	5.23	2.9
4	Bachman Branch	Midway Road	5.6	2.6	31.78	5.64	2.6
5	Joe's Creek	State Highway 114	5.6	2.9	30.99	5.58	2.9
6	Duck Creek	Garland	13.9	6.3	13.80	3.72	23.5
7	Big Fossil Creek	Haltom City	18.9	10.1	16.10	4.01	47.6
8	Village Creek	Handley	28.0	14.0	16.79	4.10	95.6

NOTE: The weighted main stream slope is somewhat time consuming to determine. Comparable results are obtained by use of a slope index method currently used by the U.S.G.S. (2) The index is computed by determining the difference in elevation between two points that are 85 and 10 percent of the main channel distance upstream from the point of interest, and then dividing by the channel distance between the two points.



BASIN LAG vs. BASIN CHARACTERISTICS
(AFTER EAGLESON, 1962)

FIGURE 2

urban classification into the following general categories (figure 3): (1) Cultivated, some urban, no storm sewers; (2) More urban, some storm sewers, no channel improvement; (3) Extensive urban, storm sewers, no channel improvement; and (4) Extensive urban, storm sewers, considerable channel improvement.

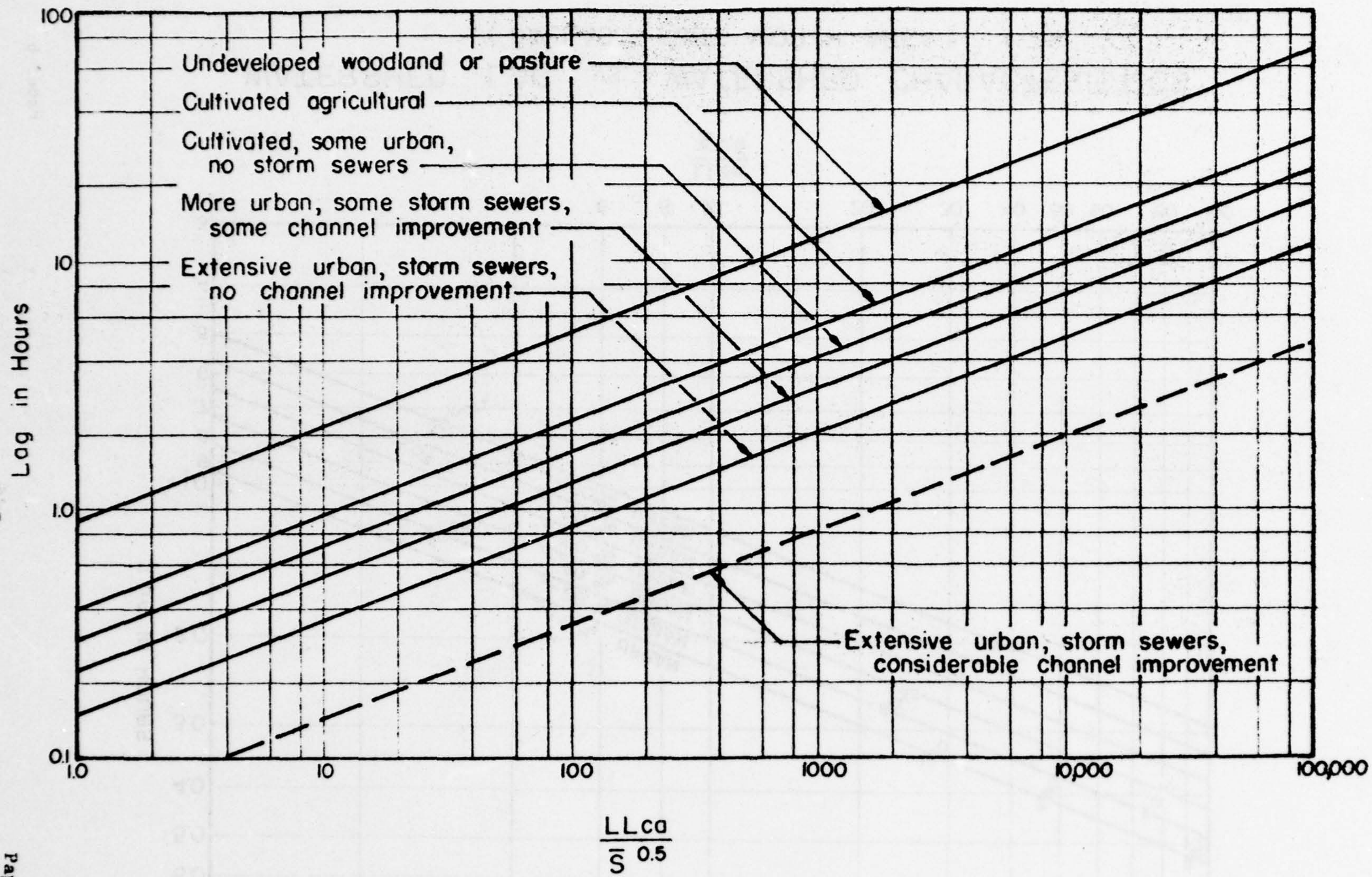
A plot of the data from the Fort Worth-Dallas study (from tables 2 and 3) is shown on figure 4. Lines drawn through these points, parallel to those of the referenced studies, indicate a family of curves whose scatter can be at least partially explained by the degree of urban development. The upper numbers noted by each point are the watershed numbers* and the lower numbers are percentages of the watershed areas which are developed urban areas. The determination of urban areas was made by measuring the indicated urban area from recent U. S. Geological Survey 7.5-minute quadrangle maps (or from more recent aerial photographs, where available).

No estimate was made of the degree of coverage by storm sewers or of the miles of channel improvement and no estimate was made of the amount of impervious area included in each urban area.

The curves indicate that in the Fort Worth-Dallas area a complete change from a rural to a fully developed urban condition would reduce the lag (t_p) for a watershed by about fifty percent.

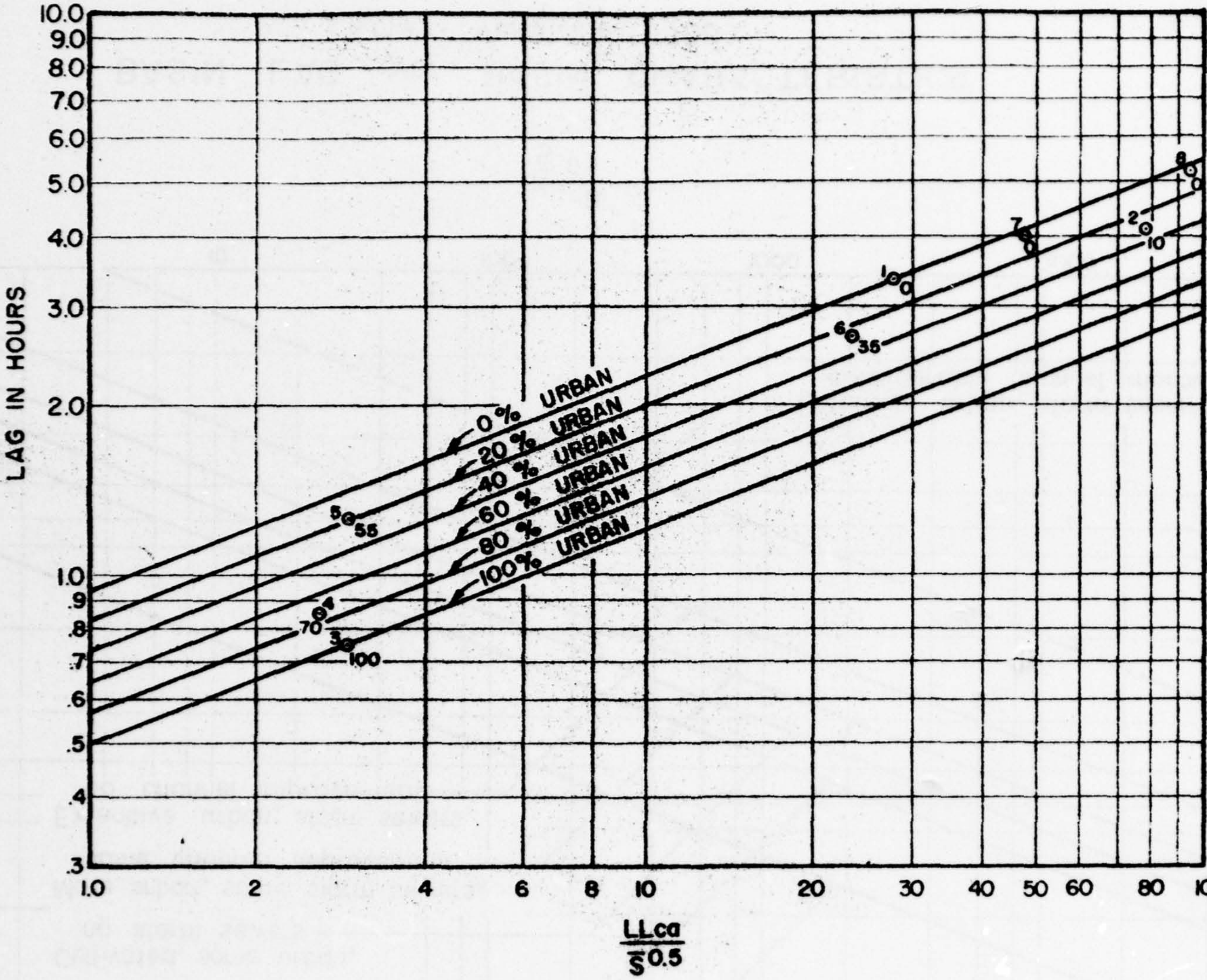
The average t_p (hours) was plotted versus the average q_p (cfs/sq mi) for each of the watersheds investigated (table 2). The construction of the curve (figure 5) relating the two variables is such that at any

*Referenced to tables 1, 2, and 3.



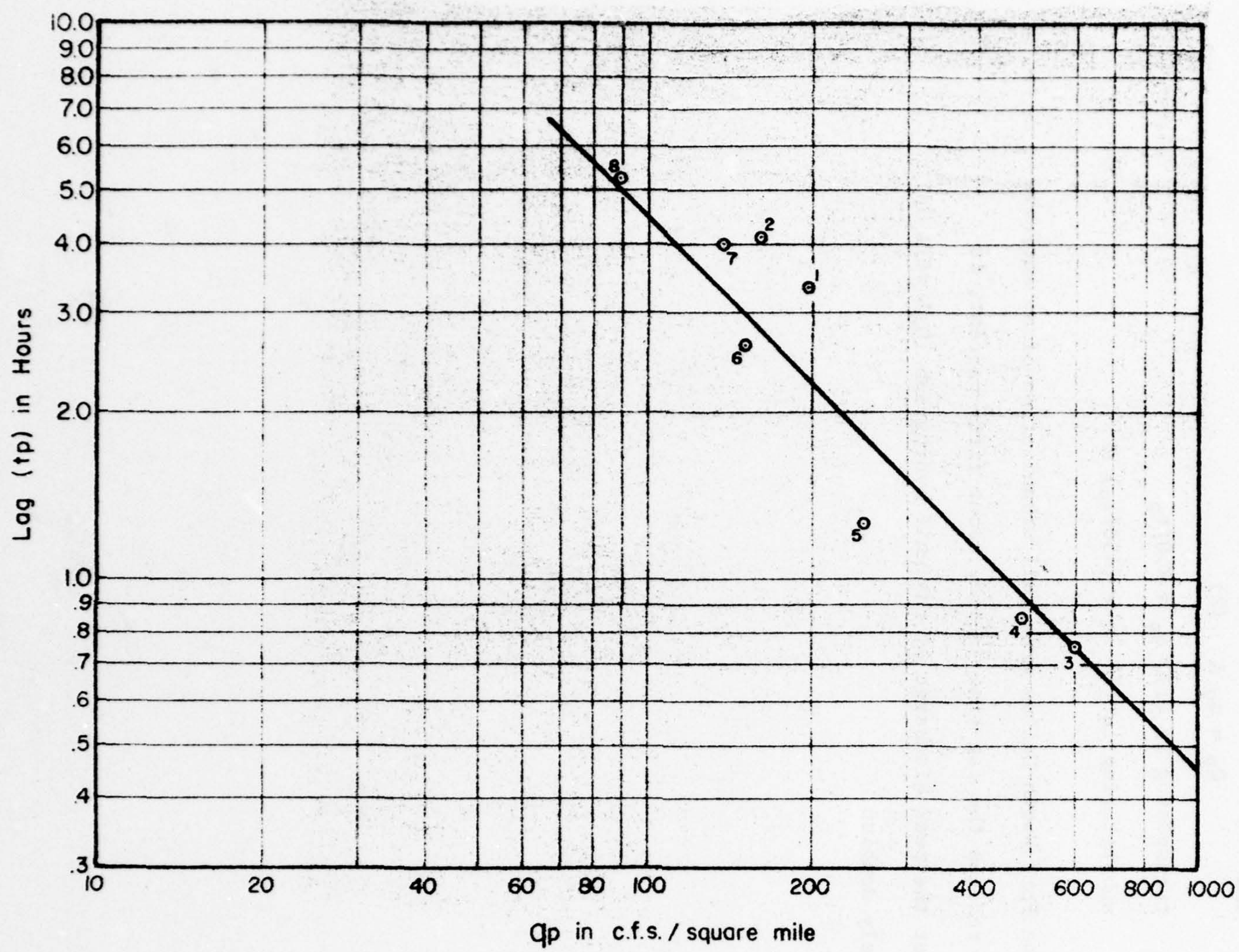
BASIN LAG vs. BASIN CHARACTERISTICS
(After VanSickle, 1968)

FIGURE 3



WATERSHED LAG vs. WATERSHED CHARACTERISTICS
(DALLAS - FORT WORTH AREA)

FIGURE 4



WATERSHED LAG vs. UNIT HYDROGRAPH PEAK

(DALLAS - FORT WORTH AREA)

FIGURE 5

point, the product of t_p and q_p is a constant, or:

$$t_p \, q_p = 450 \qquad (1)$$

and

$$Q_p = q_p \, A \qquad (2)$$

transposing in (1), $q_p = 450/t_p$

substituting in (2) $Q = (450/t_p)\, A$

Since in the Fort Worth-Dallas area full urbanization of an entirely rural area reduces the lag time (t_p) by about fifty percent, it follows that the peak discharge of the unit hydrograph (Q_p) will be approximately doubled.

CONCLUSIONS

The relationships presented may be used to develop synthetic unit hydrographs for ungaged areas in the Fort Worth-Dallas area. The method accounts for differences in urban development on adjacent areas and may be used to predict the effect that urban development might have on a given area. Preliminary results indicate that the lag relationship may also be valid in other parts of north-central Texas and further investigations will be made.

Additional analyses will be required to improve the relationships developed, to isolate the effect that channel improvement produces, and to develop an improved method of determining the unit hydrograph peak.

Improvement might also be attained by using a more sophisticated approach in determining the degree of urban development.

For the stated purpose of developing flood peaks for flood plain management studies for ungaged areas, where funds or study time are both in short supply, the present method appears to offer reasonably accurate results.

LIST OF REFERENCES

1. Floods of April 28, 1966, in the Northern Part of Dallas, Texas, Mills and Schroeder, U. S. Geological Survey Open File No. 96, December 1966.

2. Compilation of Hydrologic Data, Austin, Texas, 1968, U. S. Geological Survey, Water Resources Division.

3. Hydrology for Engineers by R. K. Linsley, Jr., M. A. Kohler, and J. L. H. Paulus, McGraw-Hill Book Co., Inc., New York, 1958.

4. Peter S. Eagleson, Proceedings A.S.C.E., Vol. 88, No. HY2, March 1962, Part I.

5. Donald VanSickle, "Experience with the Evaluation of Urban Effects for Drainage Design," Water Resources Symposium No. 2, Center for Research in Water Resources, University of Texas, 1969.

SYNTHETIC UNIT HYDROGRAPH RELATIONSHIPS -
TRINITY RIVER TRIBUTARIES
FORT WORTH-DALLAS URBAN AREA

Discussion

Question, Mr. Beard: I notice that so far the New England Division is the only one that makes special studies of rainfall frequency while other offices appear to use Weather Bureau published values. What is the practice of the Fort Worth District?

Reply, Mr. Nelson: The Fort Worth District utilizes data from USWB TP-40, including the depth-area-duration data included in that paper to adjust the rainfall for changes in drainage area. We do not use constant loss rates to apply to storms of differing frequency. We vary the loss rates, applying higher rates to the storms of lower frequency. This is an attempt to produce synthetic frequency curves which have characteristics similar to those of frequency curves derived from streamflow records in the same general area.

Comment, Mr. Beard: It appears that there is difficulty among the general public and even many engineers in appraising the significance of a 100-year or standard project flood. Perhaps it would be of significance in urban areas to consider the probability of having a flood problem in a 25-year period (a moderate length of structure life). In the case of the 100-year flood, we might say about one chance in five, which does not sound nearly as impressive as "the 100-year flood".

Reply, Mr. Nelson: This type of presentation should prove useful. We will try to get similar statements included in our flood plain information reports and used in presentations to local interests.

DISCUSSION OF SOME ASPECTS OF URBAN HYDROLOGY METHODOLOGY

by Clarence W. Timberman[1]

INTRODUCTION

The distinctive hydrologic characteristic of urban hydrology might be said to be the change in runoff response of an area as a function of its development. Thus the hydrologic analyst is faced with evaluating the effect of various physical changes in the area. A great deal of work has been done, and much is available in the literature in the general area of factors affecting runoff production. Any of this which tends to explain underlying relationships of cause and effect is applicable to urban hydrology. The application of usual statistical analysis methods is made more complicated in many cases by changes taking place during the period of record. Of course, comparison of statistical analysis from long-time rather stable urban areas to otherwise similar areas must be considered a prime source of information. Since changes in basic basin characteristics are involved it seems advantageous to use methods that, to the greatest extent possible, are developed from rational analysis of basic basin physical factors. The common unit hydrograph type of analysis based on reconstitution of specific flood events and correlation studies of unit hydrographs and basin characteristics would thus appear to be the most productive type of analysis since the unit hydrograph itself reflects an integration of basin runoff characteristics. It is recognized that there are limitations and weaknesses to this approach such as possible non-linearity of unit hydrographs for different magnitudes of floods. Procedures can possibly be refined if accuracy of data warrants by variations depending on the amount of current period and antecedent periods runoff.

It is desirable for design and other purposes to relate crest flood flow to frequency. The application of unit hydrograph methods used with statistical depth-area-duration rainfall data provides a means of doing this. Caution must be used in the application of loss or runoff factors as average condition factors based on average antecedent precipitation must be used to produce valid results. The procedure can be considered a mathematical model and as in a physical model it should be verified against known performance. This can be done in a basin that has both analysis data and a stream gage record of flow from which a frequency curve can be computed.

[1]Hydraulics and Hydrology Section, Missouri River Division

CHOW METHOD

The material presented in Appendix I to this paper describing the "Chow" method of analysis is the best description of this method known to this writer. It is taken as verbatim excerpts from a publication dated October 1965, prepared under contract to the Kansas City District of the Corps of Engineers by the Center of Research, Inc., Engineering Science Division, University of Kansas. After publication copies were distributed to Corps of Engineers Offices, but surplus copies are now exhausted. Some extra copies of the excerpts (Appendix I) have been reproduced and are available upon request. The distinctive feature of the method is the determination of the most critical length of rainfall duration for a particular frequency. Essentially the basin characteristics as reflected by the unit hydrograph itself are used to determine the critical period.

Although the material is not presented with urban hydrology in mind, the method is considered applicable. It is essentially a mathematical model method which simulates a basin's response to rainfall. For use in urban hydrology the loss factor would be suitably modified to account for typical urban infiltration rates and unit hydrograph lag factors would have to reflect effects of urban conditions. These factors would ideally be obtained by analysis of urban area data, but could also be estimated by rational adjustment of other data. Although quantitative studies have not been completed for areas in this Division, quantitative analysis of available data indicates that the effect of urbanization on time to peak of flood hydrographs can only partially be accounted for by a lag factor based on channel length and slope. An additional factor reflecting channel efficiency is believed to be required. Studies of some areas have been made using an unsteady flow routing program using basic channel cross sections and "n" value data. This can be done either for an existing condition or for a proposed design condition. It is made practical by the recent development of efficient computer programs to solve (to various degrees of approximations) the basic differential equations of flow. An observation from these studies is that channel efficiency **can vary greatly with the magnitude of flow i.e. for instance, a channel may be very efficient up to a certain natural or design flow but be partially obstructed for higher flows (bridges, road embankments, etc.). There can be major storage effects resulting from such flow restrictions that can produce changes of 100% or more in time to peak values and corresponding changes in flow values.**

It is believed that the methods described in the attached Appendix can provide a useful design tool. For use in urban areas, relationships such as shown in Figures 12, 23, and 24, Appendix I, would have to be suitably modified. The equations or graphs for determining t_p should also be modified by an additional factor reflecting channel efficiency.

The hydrograph shape factor as reflected by Figure 24, it is believed could justifiably have a shorter recession limb (less storage in the channel) and resulting higher peak flow values.

FLYNT-HORNER METHOD

An alternate to the non-dimensional unit hydrograph procedure (see Step 12, page C-2, Appendix I) developed in the Missouri River Division Office is to use a computer program to produce a unit hydrograph mathematically given time-to-peak from beginning of rainfall excess and a shape factor consisting only of total length of base of the hydrograph; that is, for a given time-to-peak the magnitude of the peak varies inversely with base length. This program can be used in lieu of the final step described in the Chow method, but is also a valuable procedure in itself. The method is based on one described by F. L. Flynt and W. W. Horner(1).

It has been found that the ratio of length of rising limb to recession limit of unit hydrograph over large regions and a large range of size of areas is remarkably stable. If this ratio is taken as a constant then only 1 parameter (time-to-peak) in addition to drainage area is needed to derive the unit hydrograph. This method is useful with Snyder method described in Corps of Engineers manuals. T_R is computed as in the manual. Selection of length of base (or limb ratio) is equivalent to selection of a C_p coefficient. W-75 and W-50 values can be used as a check on hydrograph shape.

SUGGESTIONS FOR IMPROVEMENTS AND FUTURE NEEDS

The greatest need is considered to be for additional and more detailed field data to test and verify methods and establish relationships. The greatest weakness in the computational procedure as described in Appendix I lies in the rainfall runoff relationship. It is believed that rather than applying loss rates or factors to total drainage areas, the area should be divided into a number of zones. These zones would not be contiguous in area, but represent the proportion of the total area having some range of infiltration capacity. Also more attention needs to be given to the effect of rainfall intensity. Also to be noted are two papers by F. A. Huff published in the Water Resources Research (2,3). It is believed that relationships such as developed in these papers can be used to improve computation techniques although it is not within the scope of this paper to discuss specific procedures.

Use of computers now available make more detailed computations feasible. Breakdown into smaller sub-areas and shorter time increments is practicable and combination of refined routing procedures with unit hydrograph analysis can provide for non-linear and basin storage type effects.

SUMMARY

In summary it is considered that the unit hydrograph mathematical modeling procedure appears to be a suitable method of analyzing urban area runoff providing that:

a. The unit hydrograph characteristics such as; time to peak, peak Q cfs per Sq. Mile, and recession factor can be correlated to physical basin characteristics such as; slope, basin shape, basin storage, and channel and overland flow efficiency.

b. Loss rates, or runoff factors, can be determined with sufficient accuracy and properly related to rainfall duration, season, API, rainfall intensity, etc.

c. If results are related to frequency, that the proper critical length of rainfall period has been used.

REFERENCES

1. Horner, W. W. and Flynt, F. L., "Relation Between Rainfall and Runoff From Small Urban Areas", Transactions, Volume 101, 1936, American Society of Civil Engineers.

2. Huff, F. A., "Time Distribution Characteristics of Rainfall Rates", Water Resources Research, April 1970.

3. Huff, F. A., "Spatial Distribution of Rainfall Rates", Water Resources Research, February 1970.

APPENDIX I

This Appendix contains excerpts from "Hydrologic Design for Small Watersheds", by Robert L. Smith, George S. Clausen, John A. Henderson, (Report prepared under provisions of U. S. Army Engineer Contract DA-23-028 CIVENG-64-821) by The University of Kansas, Center for Research Inc., Engineering Science Division, Lawrence, Kansas; October 1965. These excerpts pertain to Chow method of unit hydrograph design procedure.

INTRODUCTION

The field and office studies described herein were initiated to answer certain hydrologic design questions encountered by the U.S. Army Corps of Engineers. The specific problems posed are associated with the development of a combined agricultural levee and channel stabilization program along the Missouri River from Sioux City, Iowa, to St. Louis, Missouri. Program installation necessarily leads to appreciable alteration of the local drainage system. Large acreages, originally meander wastelands, are being converted to productive agricultural areas. Surface runoff from adjacent watersheds must be conveyed across these areas and through the levee. In some instances these interior watersheds consist almost entirely of old Missouri River floodplain, others are primarily typical hill or upland watersheds, and still others represent a combination of floodplain and hill area

Cost of the necessary drainage structures will aggregate many millions of dollars. Faced with a need to avoid costly overdesign, and an almost complete lack of prior data for the unique physiographic conditions involved, the Kansas City District Office of the Corps of Engineers entered into a cooperative arrangement with the U.S. Geological Survey for the collection of hydrologic data on two adjacent, but vastly different watersheds in the vicinty of Atchison, Kansas. These data efforts were initiated in late 1960 and continue to date.

This report is devoted to analysis of the data collected through 1964, and to formulation of a tentative design procedure for hydrograph synthesis based upon evaluation of this initial data and review of the prior literature. Emphasis throughout is directed toward meeting the applied needs associated with the aforementioned Missouri River program - namely - the resolution of a design procedure which (a) is based on sound and acceptable hydrologic principles, (b) requires a minimum of field data, (c) is applicable to a wide range in physiographic conditions, and (d) is relatively simple to apply.

Following a description of the experimental areas and a summary of the data obtained thereon, there is presented a brief review of basic hydrologic design concepts and the application of these concepts to several of the more common design methodologies. The experimental data is then discussed in light of these concepts and methodologies, and a tentative

design procedure, based on the work of Chow (1962) with certain modifications resulting from analyses described herein, is outlined. Preliminary use of the recommended methodology by the Kansas City District Office indicates its application to typical design situations will result in a more consistent and economical design than has been possible heretofore.

DESCRIPTION OF EXPERIMENTAL AREAS

General Location

Throughout this report reference is made to the relative hydrologic response of two small watersheds which were selected for special field study by the Kansas City District Office, U.S. Army Corps of Engineers. These two experimental areas are situated in Atchison and Doniphan Counties, Kansas adjacent to the Missouri River and immediately upstream of Atchison, Kansas. The relative location of the two watersheds and their respective drainage boundaries are shown in Figure 1. Inititally, both areas were part of Independence Creek. However, during the construction of Levee Unit 440-R the lower area (formerly a portion of the common floodplain of Doniphan Creek, Independence Creek, and the Missorui River) was effectively separated from the upland drainage system. The upper area (officially known as Doniphan Creek) continues to empty into Independence Creek. For the comparative purposes of this report the lower area will be referred to as the Floodplain Watershed and the upper area (Doniphan Creek) will be known as the Hillside Watershed.

Physical Characteristics

The Hillside Watershed drains a total of 2610 acres at the stream gage. The Floodplain Watershed is believed to have a contributing bottomland area of 4000 acres at the levee outlet. A very small area (approx. 30 acres) of adjoining hillside also drains into the upper end of the floodplain via a conduit under the road at the base of the bluff.

The Hillside Watershed ranges in elevation from 800 feet above mean sea level at the outlet to 1050 feet above mean sea level at the uppermost point. The Floodplain Watershed is essentially flat, but ranges in elevation from 780 feet above mean sea level at the lower end to 795 above mean sea level at the upper end.

Both drainage areas are roughly rectangular in shape with the long axis of each oriented in a north - south direction. The Hillside Watershed has a typical dendritic drainage pattern and the length of the longest watercourse is 3.5 miles. The Floodplain Watershed is drained by the

old channel of Independence Creek and several drainage ditches. Its drainage system is being improved from year to year as the land is being converted from waste to intensified farming.

There are four major soil types in the Hillside Watershed and six major types in the Floodplain (Soil Conservation Service, 1964). The hydrologic rating of these soils and the approximate per cent of area covered by each are shown in Table I.

Table 1 - Soils in Experimental Areas

Soil Type (Uncorrelated)	Hydrologic[(1)] Rating	Approximate Per Cent of Area
Hillside		
Labette Silty Clay Loam	C	26
Ladoga Silt Loam	C	18
Marshall Silt Loam	B	37
Monona Silt Loam	B	19
Bottomland		
Eudora Silt Loam	B	18
Kannaboe Silt Loam	B	20
Onawa Clay	C	33
Onawa Silty Clay	C	10
Sarpy Fine Sandy Loam	A	5
Wabash Clay	D	14

(1) Hydrologic ratings are relative with runoff greatest in D group and least in the A group.

The Hillside Watershed exhibits geology typical of northeast Kansas showing the effects of two major glacial advances, the Nebraskan and the Kansan, in fairly recent geologic time. These glaciers left accumulations of till and outwash material which is as much as 300 feet thick in places. After the glaciers retreated wind-blown silt (loess) was deposited over the till. Wind and water have since eroded much of the loess mantle, but it still exists to a thickness of 30 feet in some places. The Floodplain area consists of river-transported sediments ranging from very fine to very coarse gravel. The entire area is underlain by sedimentary rocks of the Upper Pennsylvanian Douglas Group.

However, in the Hillside Watershed the glacial material exists to such a thickness that it controls the topography and the usual bedrock-control is absent.

Climatic Conditions

The area lies in the path of alternate masses of warm, moist air moving north from the Gulf of Mexico and currents of cold comparatively dry air moving down from the polar regions. Consequently, its weather is subject to frequent and often sharp changes, usually of short duration.

The average temperature in January, the coldest month of the year, is about 29 degrees Fahrenheit while during the warmest month of the year, July, the temperature averages about 79 degrees Fahrenheit. The first killing frost usually occurs in the first or second week of October.

The mean annual precipitation at nearby Atchison, Kansas is 34.53 inches and more than 70 per cent of this occurs during the six months April through September.

Instrumentation

The Kansas City District, U.S. Corps of Engineers, in cooperation with the U.S. Geological Survey's Surface Water Branch, began to collect field data on rainfall and runoff in the project area in 1960. The area was equipped with five recording and four non-recording rain gages along with three recording stream gages. The location of these gages is shown in Figure 2.

The two tipping bucket rain gages located at the stream gaging stations were used primarily to reconcile time scale problems with the stream recorders.

The outlet from the Floodplain area is through three gate-regulated, 72-inch culverts. One of the stream recorders at this location was used to measure the headwater while the other measured tailwater level.

The Kansas Geological Survey, in cooperation with the U.S. Geological Survey, established eighteen groundwater observation wells in the Floodplain area during the summer of 1963 (see Fig. 2). Two of these wells were equipped with recording gages for most of the calendar year 1964.

In addition, six wells previously installed by the Corps were monitored. Also, during calendar '64 the Corps of Engineers collected periodic soil moisture samples at four locations within the Floodplain basin. This sub-surface information, however, was considered too fragmentary for analytical needs, and no quantitative reference to this data is contained herein. The groundwater data was most helpful in resolving qualitatively the hydrologic cycle of the Floodplain.

is $\pm$ 30 per cent it incorporates a correction coefficient.

The Chow Method

During the period 1952-62 extensive studies of methods for designing waterway openings were undertaken at the University of Illinois under the joint sponsorship of the University, the Illinois Highway Commission and the U.S. Bureau of Public Roads. This work culminated in the publication of a report (Chow 1962) which outlined a new peak flow design methodology. The method relates the basic concepts of unitgraph theory to a computational process almost as simple as that afforded by the rational formula. The methodology merits attention, and is now being utilized in France, Cyprus and Louisiana as well as Illinois. (personal correspondence - July 1965)

The Chow method relies on the SCS approach for determination of the rainfall-runoff relationship. As will be shown later this tends to minimize the effectiveness of the system. Our immediate interest, however, lies in Chow's treatment of hydrograph theory in arriving at an estimate of peak flow.

Chow utilized the basic concepts of S-hydrograph and unitgraph theory to calculate the peak discharge for a steady rainfall excess of x inches per hour for t hours. Each peak discharge so determined was expressed as a decimal fraction of the equilibruim peak that would occur if the rain continued indefinitely. This ratio is defined as Z, the peak reduction factor, and for any watershed can be plotted as a function of the dimensionless ratio of t/T_L where t is the duration of rain and T_L is the basin lag or time from center of mass rainfall to center of mass runoff. This peak reduction factor represents the effect of basin storage which is not evaluated in the normal application of the rational formula.

As noted earlier T_L is an awkward parameter to measure. Chow therefore adopted as a measure of the detention effect the time t_p which is defined as the period of rise on an instantaneous unit hydrograph. The instantaneous unit hydrograph is a hypothetical unitgraph whose duration of rainfall excess approaches zero (Dooge 1959). The integration of an area covered by the instantaneous unit hydrograph is proportional to the ordinate of an S-hydrograph. Therefore, the ordinate

of the IUH is proportional to the slope of the S-hydrograph and the t_p can be determined by plotting an S-hydrograph and measuring the time to the point of inflection (Figure 11). Finally, Chow reasoned that for small basins t_p as measured above would be independent of rainfall duration and be closely equivalent to the time from center of mass of rainfall to peak of hydrograph. Using these assumptions, and data obtained from 20 small midwestern basins he developed an average Z vs. t/t_p relationship.

To convert these concepts and assumptions to a design procedure Chow proposed the formula Q peak = XZA

where X = average intensity of rainfall excess in inches/hour
Z = peak reduction factor
A = drainage area in acres
Q = discharge in cfs

The rainfall excess intensity factor can be related to storm frequency and thus defines a specific design criteria. The peak reduction factor provides a measure of basin storage effects for any selected duration of rain. The produce (XZ) of these two factors allows determination of an optimum storm period. The concept is not without its difficulties. There still remains the problem of converting rainfall to runoff, the need to develop a realistic expression for estimating t_p, the question of runoff frequency versus storm frequency, the possible variance to be encountered among watersheds in the Z vs. t/t_p relationship, and the time distribution of the hydrograph associated with a calculated peak flow. These and related matters are discussed in the section on review of experimental data.

alternative of using a constant curve number produces a situation where the calculation of X and Z, as used in the Chow equation, are not for comparable times t. Where local rainfall-runoff coaxial relations are not available use of the appropriate SCS Condition III curve is recommended for calculation of approximate loss rates for periods up to two hours duration. The information so obtained can be used as a guide for preparation of a loss rate curve similar to Figure 14.

Chow recommended that the rainfall, as selected from a rainfall frequency curve be increased six per cent to allow for non-uniformity in the rainfall. Though the problem of non-uniformity is most real the indicated six per cent increase in rainfall does not appear critical. Far more important is the selection of the basic moisture condition (API or SCS wetness condition). For example, Condition III runoff for typical midwestern cover conditions varies from 300 to 160 per cent of Condition II runoff as precipitation increases from 2 to 6 inches.

Time to Peak

The several storm hydrographs for the two watersheds were converted to S-hydrographs and a t_p determined for each storm. For the storms studied, t_p values ranged from 0.5 to 3.0 hours on the Hillside and 4.0 to 24.0 hours on the Floodplain. Average values were 1.3 hours on the Hillside and 13.5 hours on the Floodplain. Only two of the observed Floodplain hydrographs had t_p values less than 9.75 hours. These were the storms of September 3, 1961 and May 20, 1962. Subsequent studies indicated that the former might be influenced by backwater, and the peak on the latter (a very small volume storm) probably did not reflect a reasonable percentage of contributing area.

These observed hydrographs at the Floodplain gage were necessarily the outflow graphs from the channel ponding area upstream of the gate structure. Discounting any normal channel storage in this reach, and excluding the three small volume storms and the aforementioned questionable hydrograph of September 3, 1961, this pond storage averaged 8.5 per cent of the storm runoff at the time of observed peak. To ascertain the maximum possible effect of this ponding on hydrograph peak and time of peak, several of the hydrographs

were back-routed with no allowance made for normal channel storage. These routings indicate the observed t_p values should be decreased an average of 3.5 hours and the observed peaks increased an average of 12 per cent. Thus, it is concluded that the mean t_p should be adjusted downward to approximately 10.0 hours. Mean dimensionless S-hydrographs (per cent of equilibrium q vs. t/t_p) were developed for the Hillside, Floodplain as observed, and Floodplain as corrected for ponding effect and these are shown in Figure 19.

Chow, after analyzing the data from 20 small midwestern watersheds, recommended time to peak be determined by the equation

$$t_p = 0.00236 \; (L/S^{0.5})^{0.64}$$

In the foregoing L is the channel length in feet as measured up the main watercourse to the divide and S is the slope in per cent. The latter is determined by plotting the stream profile and fitting a straight line through the gaging station so that the area between the line and the profile lying below the line is equal to that lying above it. t_p is measured in hours.

Application of the foregoing equation and each of the standard methods outlined earlier give hydrograph times comparable to those observed on the Hillside. Without exception, all methods cited underestimate appreciably the time scale for the Floodplain area. The long lag period observed on the Floodplain is easily explained from a qualitative viewpoint. Reasonable quantification of the relationship is necessary.

Morgali's analysis of two-dimensional overland flow indicated that time was a function of $L^{0.6}/S^{0.4}$ where the exponents have been rounded to the nearest tenth. Question arises as to whether on small watersheds the time parameter is controlled primarily by overland flow considerations as suggested by Hickok et. al. Accordingly, Chow's data for t_p was plotted against $L_o^{0.6}/S_o^{0.4}$ where L_o equals overland flow length calculated by use of the formula proposed by Horton (1945) and S_o equals average land surface slope. The resulting plot was not definitive.

Inasmuch as Morgali's basic solution of the continuity and momemtum equations should be equally applicable to channel flow, there is incentive to relate t_p to L and S using the exponential powers he suggested. Accordingly, the data presented by Chow was plotted against various combinations (designated herein as lag factors) of length and slope parameters. These included those terms previously defined and basin width (area divided by L). Regression equations were determined as follows.

Lag Factor (LF)	Regression Equation	Standard Error of Estimate in Log Cycles	Correlation Coefficient
$L/S^{0.5}$	$t_p = .00107\ (LF)^{.576}$	.178	.892
$L^{0.6}/S^{0.4}$	$t_p = .00562\ (LF)^{.893}$	.170	.902
$L^{0.6}/S^{0.4} + L_o{}^{0.6}/S_o{}^{0.4}$	$t_p = .00226\ (LF)^{1.026}$	.154	.921
$W^{0.6}/S^{0.4} + L_o{}^{0.6}/S_o{}^{0.4}$	$t_p = .00370\ (LF)^{1.020}$	.176	.894
$W^{0.6}/S^{0.4}$	$t_p = .00473\ (LF)^{1.035}$	.150	.926

In all cases where exponents of 0.6 for the typical length term and 0.4 for the slope term were utilized the slope of the regression line approximates unity. However, none of the equations when extended provide a realistic explanation of the Floodplain t_p. Since the lag factor for the Floodplain is appreciably larger than that encountered on the other watersheds, it would appear possible that the difficulty results from a changing t_p vs. lag factor relationship.

To check the foregoing, available hydrograph data for two larger watersheds (Lyons Creek, Kansas - Drainage Area 230 square miles and Wakarasa River, Kansas - Drainage Area 425 square miles) whose lag factors exceed that of the Floodplain were utilized to develop t_p values. This data tended to

confirm the change in the t_p vs. LF relationship. Figure 20 represents a plot of t_p vs. two different measures of lag, and includes data from Chow's watersheds plus the two experimental areas and the two larger Kansas basins.

In each case two relation lines are shown. The first is based solely on Chow's data, the second replaces Chow's four smallest watershed with the four Kansas streams. These plots indicate a slowly changing relationship between t_p and LF which can be approximated by two straight lines for the range of data tested, and the equations shown on Figure 20 are recommended.

It is recognized that for preliminary design purposes available data do not always allow easy or convenient determination of the lag factor (LF) of a watershed. Often drainage area is the only watershed characteristic initially available. It is possible to express the foregoing relations in terms of drainage area. However, additional field data on the physiographic characteristics of other floodplain areas is needed before it can be determined if truly useful approximations for that terrain can be made solely on the basis of drainage area. The following paragraphs will serve to explain.

As previously noted, there are sound mathematical reasons for relating hydrograph time characteristics to watershed length and slope parameters. These parameters, however, are not independent of area. For example, each of the two lag factors used in Figure 20 was plotted against drainage area. The resulting curves had correlation coefficients approaching unity. Moreover, a check with other available data indicated the resulting equations were applicable to Kansas watersheds as large as 1000 to 2000 square miles in size. The equations so determined are:

$$W^{0.6}/S^{0.4} = 12.6\ A^{.375} \qquad (1)$$

$$L^{0.6}/S^{0.4} = 12.0\ A^{.47} \qquad (2)$$

where area is in acres and other terms as defined earlier.

Substitution of the foregoing equations for A in the t_p equations of Figure 20 results in the following: For areas in excess of 300 acres.

$t_p = .024A^{0.555}$ when equation (1) is used and (3)

$t_p = .026A^{0.555}$ when equation (2) is used (4)

For areas less than 300 acres

$t_p = 0654A^{.388}$ when equation (1) is used and (5)

$t_p = 0517A^{.420}$ when equation (2) is used (6)

Equations (3) through (6) are only valid for watersheds whose general morphology is consistent with equations (1) and (2). Thus, they are not applicable to floodplain areas unless there is some way of converting to an equivalent size "normal" watershed. Figure 21, which shows the general relationship of lag factor vs. drainage area (Equation 1) for a representative group of watersheds, will serve to illustrate. It will be noted that the 4000 acre Floodplain area exhibits the lag factor characteristics of a "normal" area ten times as large. If it is assumed that this one floodplain is typical of all floodplain areas, a lag factor versus drainage area relationship might be visualized as shown by the dashed curve on Figure 21, and equations similar to those shown in (3) through (6) could be calculated for use in preliminary design of floodplain areas. However, development of such equations should await determination of lag factor data for additional sized floodplain areas so that the true location of the dashed line on Figure 21 can be determined.

An option to developing a separate set of preliminary design equations for each individual curve that might be developed on Figure 21 is to use the so-called "equivalent" area in equations (3) to (6). Thus, in the case of the Floodplain watershed an equivalent area of 40,000 acres would be used. Figure 22 is a plot of the observed t_p values versus equivalent area for Chow's watersheds where the latter has been determined from equation (1) using measured values of W and S. Finally, the reader is reminded that this entire discussion relating t_p to A is prompted by problems arising at the preliminary design stage when drainage area may be the only known watershed characteristic.

Similarly, the individual parameters of S and L can be related to A for the watersheds studied. It is of interest to note that when surface slope is expressed in terms of A and substituted in the equation proposed by Hickok, et. al., for small western watersheds of less than 1000 acres, t varies

as approximately the 0.4 power of area. Also, if L, L_c, and S are expressed in terms of A and substituted in the relationship

$$T_p = C\ (L\ L_c/S^{0.5})^{.38} \qquad \text{(Corps of Engineers 1946)}$$

a widely used expression for larger areas, T_p is found to vary as approximately the 0.6 power of area.

Table V summarizes the appropriate physical parameters and observed t_p values for the watersheds utilized in this study.

Peak Reduction Factor Z

Any attempt to relate observed peak discharges to existing methodologies proved most fruitless on the Floodplain owing to the problems associated with the time scale and the unusually high attenuation resulting from the unique geomorphology of the area.

The SCS hydrograph procedures for determining peak discharge provided reasonable correlation with observed data on the Hillside. Similarly, the Bureau of Public Roads procedure which estimates peak discharge on a frequency basis provides a reasonable peak estimate for the Hillside. The method presently used by the Kansas City District Office, Corps of Engineers, fails to compensate for storage attenuation. Although the Chow approach, with its reliance on uniform rainfall, is not readily amenable to direct comparison with observed storms, the latter can be utilized to determine the general shape of the S-hydrograph and the Z vs. t/t_p relationship.

The respective mean S-hydrographs for the experimental watersheds were lagged for varying times t and a Z (peak reduction factor) vs. t/t_p relationship developed for each watershed. These curves for the Hillside and the Floodplain, as corrected for pondage at the gate structure are shown in Figure 23 together with an average curve developed by Chow from a plot of individual storm values for the 20 watersheds he studied. Chow arbitrarily made $Z = 1.0$ at $t/t_p = 2.0$ in keeping with his assumption that $t_p = T_p$. However, it is equally convenient to develop the t_p vs. T_p relationship that conforms to the individual S-hydrographs. The latter are shown in Figure 24.

TABLE 5

WATERSHED CHARACTERISTICS[1]

Name of Watershed	Drainage Area (acres)	Length (ft.)	Width (ft.)	Channel Slope (%)	t_p (hrs.)
W-1 Edwardsville	27.22	1,650	811	1.51	0.315
W-4 Edwardsville	289.8	19,800	2,190	1.29	0.443
W-1-A, Monticello	82.0	3,530	1,685	0.66	0.576
W-1-B, Monticello	45.5	2,053	721	0.54	0.436
West Salem	969	6,600	4,250	0.51	1.872
Madden Creek	992	6,440	4,890	0.58	1.809
Edwards County					
Hurricane Creek	92.2	3,500	2,110	0.60	0.625
Trib., Witt					
W-97, Coshocton	4,580	52,800	7,640	0.53	1.900
W-183, Coshocton	74.2	3,400	1,020	0.65	0.240
W-196, Coshocton	303	9,000	2,960	3.70	0.310
W-1, Hamilton	187	5,390	1,632	1.15	0.286
W-3, Bethany	4.85	1,660	300	6.25	0.117
	4.48	1,660			
W-1, Fennimore	330	8,200	2,490	2.03	0.419
W-2, Fennimore	22.8	560	992	4.75	0.116
W-4, Fennimore	171	4,000	2,280	2.17	0.267
W-1, Colby	345	4,490	2,290	0.76	0.417
W-5, Lafayette	2.87	280	220	1.41	0.139
W-6, Lafayette	2.79	240	208	1.48	0.165
Ralston Creek	1,926	64,240	3,890	0.60	1.422
Iowa City					
W-3, Hastings	481	30,500	2,330	0.55	0.636
Hillside Area	2,610	19,500	5,840	.646	1.30
Floodplain Area	4,000	28,400	6,160	.035	13.65
Lyon Ck. nr. Woodbine	147,000	165,300	48,500	.182	16
Wakarusa R. nr. Lawrence	273,500	307,000	38,700	.164	28
Rock Ck. nr. Louisville	82,000	113,000	38,000	.322	
Salt Ck. nr. Lyndon	71,000	140,000	31,600	.106	
Turkey Ck. nr. Abilene	91,500	107,000	22,150	.188	
Verdigris R. nr. Coyville	477,000	407,000	37,600	.101	
Verdigris R. nr. Altoona	727,000	595,000	51,200	.076	
Deleware R. at Valley Falls	590,000	282,000	38,600	.123	

1) First 20 watersheds in above tabulation from data published by Chow (1962)

Chow's initial paper suggested that the z vs. t/t_p plot would shift to the right as the size of the drainage area increased. Utilizing the qualitative method of "building up" a hydrograph as outlined by Chow and the normal lag factor vs. drainage area relationships cited herein, it is possible to show that a larger area would produce a flater S-curve. However, the same procedure indicates that if a watershed has an abnormally high or low lag factor - drainage area relationship the z vs. t/t_p plot will shift accordingly. Obviously the relationship is not exact but it is proper to assume that the peak reduction factor decreases as the relative lag factor increases. Similarly, good channel conveyance would increase z and poor channel conveyance would decrease z.

Separate computations for two large watersheds (Lyons Creek, Kansas - Drainage Area 230 sq. miles and Wakarasa River, Kansas - Drainage Area 425 sq. miles) having normal to below average lag factors resulted in z vs. t/t_p relations to the left of the Chow curve thus serving to confirm the above reasoning. By way of contrast, Chow found that the average plot for 80,500 and 1200 square mile watersheds in Illinois would plot to the right of his average curve and thus concluded that positioning of the curve becomes primarily a function of area. However, these larger Illinois watersheds apparently possess above average lag factors for the data cited by Chow was based on work by Mitchell (1948) who reported that channel slope, a primary measure of lag factor, was so small as to make its determination impractical. Thus, it seems probable that part of the shift which Chow attributed to basin size might more properly be charged to the relative storage characteristics of the basins.

In view of the foregoing discussion it seems doubtful that area size would be the controlling factor within the range of small watershed design for the determination of the z vs. t/t_p relationship and the relative conveyance characteristics of the channel system, are believed to be major influencing parameters. It is concluded that the z vs. t/t_p relationships shown in Figure 23, which also serve to envelope the experimental small area data cited by Chow, can be used as an effective guide for the evaluation of z. The upper curve would be applicable to an area having either abnormally low lag factor characteristics or unusually high conveyance capability, and/or in cases where most

conservative design is desired.

Time Distribution of Hydrograph

To develop the time distribution of the runoff hydrograph the mean S-curves for both watersheds were lagged for varying times t and the resulting hydrograph for Xt inches of rainfall excess (runoff) determined. These in turn were reduced to dimensionless hydrographs with abscissa equal to t/T_R and ordinate of q/q_p. T_R is equal to $t/2 + T_p$ (See Figure 6).

A mean dimensionless hydrograph was then determined for each watershed and these, together with the standard SCS dimensionless hydrograph, are shown in Figure 24. It will be noted that the rising limb is essentially the same in all three of the dimensionless hydrographs. As the relative lag factor increases the slope of the recession limb decreases. However, the shape of the recession is usually not critical, and in actual practice the choice of the recession curve can be governed by the need to balance out hydrograph volume using the average hydrographs of Figure 24 as a guide.

CONCLUSIONS

The experimental data reviewed herein is admittedly limited as to both quantity and quality. Nonetheless, these data, when considered from the viewpoint of design needs and optional design methodologies, provide some very useful information. The general conclusions are as follows:

(1) Small area hillside streams produce volume yields comparable to those experienced from larger basins having similar climatic and physiographic environments. Hence, available volume - frequency - duration relationships obtained from larger streams can, with appropriate adjustments for hydrograph base length, be used to establish design criteria for projects requiring complete storage of hillside runoff.

(2) Total stream discharge from floodplain areas will be measurably less than from comparable size hillside areas owing to the absence of base flow and higher depressional losses in dry periods. The loss function on the floodplain remains higher than that on the adjacent hillside during wet periods, but the relative difference is not as great as has been assumed heretofore.

(3) Time scales of hydrographs from floodplain areas are comparable to those experienced on much larger conventional watersheds whose lag factor, as measured by characteristic basin length and slope parameters, is equivalent to that of the floodplain area.

(4) The combination of increased lag (item 3 above) and reasonably comparable volume (item 2 above) results in floodplain peak discharges which are appreciably smaller than those experienced on equivalent size hillside areas. Although the ratio of hillside peak to floodplain peak, when expressed on a unit area basis, varied over a wide range for the storms reported herein, it is believed that this ratio for these two watersheds is approximately 10 for storms having an equivalent risk criteria (not equal duration).

(5) The method proposed by Chow for the determination of peak discharges on small watersheds, with certain modification as outlined below and illustrated in the sample computations of Appendix C, provides a convenient and fundamentally sound approach for the calculation

of design storm hydrographs and peak discharges for either hillside or floodplain areas.

(6) The Chow methdology, as modified below, is preferable to other approaches because it can be related to a specific design criteria, is based on established hydrologic concepts, is relatively easy to apply, provides a basis for evaluating basin storage affects, and allows determination of an optimum design discharge and volume for a selected design criteria.

Specific recommendations for design methodology are as follows:

(1) Determine the area (A) of the watershed in acres.

(2) Determine the length (L) in feet and average width $(\frac{43560A}{L})$ of the watershed by measuring the distance up the longest watercourse from the point of design discharge to the divide.

(3) Plot the profile of the watercourse and determine the average slope S, in per cent, of the straight line which results in equivalent areas between the line and profile for the portions above and below the intersection of the line and profile.

(4) Calculate a rainfall vs. duration curve for a selected design frequency utilizing the data presented in U.S. Weather Bureau Technical Paper No. 40.

(5) Utilizing the information from (4) calculate the runoff for varying duration, t, of rainfall by use of the appropriate chart from Figures 12 and 13. For areas having soils, physiography, and culture significantly different from the experimental watersheds use of the standard SCS charts assuming Condition III moisture is recommended. (These are available in the SCS National Engineering Handbook; also in Chow (1962) and U.S. Dept. of Interior (1960)). In the latter case use the selected SCS curve number for preliminary estimation of loss rates for storm durations of two hours or less. For longer durations adjust curve numbers downward so that the resulting loss rate will plot a smooth curve similar in shape to those shown in Figure 14.

(6) Divide the runoff obtained in (5) by the duration of rain t and call these values X.

(7) Calculate basin lag factor, $W^{0.6}/S^{0.4}$.

(8) Calculate t_p from one of the following equations.

For lag factor less than 100: $t_p = .00473LF^{1.035}$

For lag factor greater than 100: $t_p = .000567LF^{1.48}$

(9) For various durations of rain t determine Ƶ from the appropriate Ƶ vs. t/t_p relationship of Figure 23.

(10) Calculate the product of X (Step 3) and Ƶ (Step 9) for varying times t.

(11) Multiply the largest value of XƵ (Step 10) by area A to obtain estimated peak discharge Q = XƵA. If the hydrograph is to be subjected to temporary ponding it may prove desirable also to calculate the peak Q for the time t which provides the largest volume of runoff (Step 2).

(12) Utilizing the peak discharges of Step 11 and the dimensionless hydrograph relations of Figure 24 calculate and plot the design hydrographs.

Where the basic rainfall-runoff relationship can be assumed relatively constant the procedures outlined above can be shortened somewhat by the initial computation of a series of design charts for a range of precipitation frequencies, and geographic locations along the river, and for a selected API and t/t_p vs. Ƶ relationship. Such charts would take the form of peak discharge per unit area (XƵ) as ordinate and either lag factor, t_p or equivalent area as abcissa. Finally, a family of lines representing the T_R parameter can be added by locating a number of these points on each of the XƵ vs. t_p curves.

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NOMENCLATURE

In the absence of specific definition to the contrary, symbols used in the text are as follows.

A	Drainage area in acres
API	Antecedent precipitation index taken as the sum in inches of either the prior 96 hour or 4 day precipitation
a	An empriical constant
C	Runoff coefficient in the rational formula
D	Time duration of rainfall excess
D.D.	Drainage density (length of visible channel per unit area).
e	Napierian base
f_c	Infiltration rate, normally inches per hour, at time infinity
f_p	Infiltration rate at any time t
f_o	Infiltration rate at time zero
H	Vertical difference in watershed elevation in feet measured along extension of main watercourse
I	Rainfall intensity in inches per hour
I_a	Initial abstraction or loss in inches
i	Intensity of rainfall excess, usually in inches per hour
k	Empirical constant
K	Empirical constant
L	Length of watershed measured along main watercourse extended to the divide
L_c	Length measured up main watercourse to point closest to basin centroid
L_o	Length of overland flow
LF	Defined as basin lag factor and represented by various combinations of length and slope parameters
N	Curve number in SCS rainfall-runoff relationship
n	Manning roughness coefficient
Ø	Uniform infiltration rate at which volume of rainfall excess equals volume of observed runoff
P	Precipitation in inches
Q	Normally volume rate of flow in cubic feet per second, but also represents runoff volume in inches in SCS rainfall-runoff relation
q_p	Volume rate of flow at a condition of peak discharge
S	General reference to channel slope (see text for alternative means of measuring this term). When used in SCS rainfall-runoff relation S becomes maximum potential difference between volume of precipitation and volume of runoff.

S_o	Slope of land surface
S_a	Slope of land surface
T_b	Time length of base of hydrograph
T_c	Time of concentration and/or time from end of rainfall excess to point of contraflexure on recession limb of hydrograph
T_L	Time from center of mass of runoff producing rainfall to center of mass of runoff hydrograph
T_p	Time from center of mass of runoff producing rainfall to peak of hydrograph
T_R	Time from beginning of rise to peak of hydrograph
t	General measure of time
t_{max}	Time to peak of overland flow hydrograph
t_p	Time to peak of instantaneous unitgraph
W	Average width of watershed defined as watershed area divided by L
X	Intensity of runoff (rainfall excess) generation in inches per hour
Ƶ	Ratio of hydrograph peak discharge to equilibrium discharge

APPENDIX C

Sample Computations

Design of 10-year flood for Hillside Area

Step 1: Given area = 2610 acres
Step 2: Given W = 5840 feet
Step 3: Given S = 0.646
Step 4: Columns 1 and 2 in table
Step 5: Fig. 12 gives column 3 in table
Step 6: Column 4 in table
Step 7: Basin lag factor $= \frac{W^{0.6}}{S^{0.4}} = \frac{(5840)^{0.6}}{(.645)^{0.4}} = 215$
Step 8: $t_p = .000567\ LF^{1.48} = .000567\ (215)^{1.48} = 1.6$ hours
Step 9: Column 6 in table
Step 10: Column 7 in table
Step 11: Q = XEA = (.83) (2610) = 2160 cfs

Design for API = 2, June-July

(1) Time	(2) Rainfall Amount (in.)	(3) Runoff Amount (in.)	(4) X	(5) t/t_p	(6) E	(7) XE
15 min.	1.4	.80	3.20	.16	.18	.58
30 min.	2.0	1.25	2.50	.31	.33	.83
1 hr.	2.5	1.48	1.48	.62	.54	.80
2 hr.	3.0	1.40	.70	1.25	.80	.56
3 hr.	3.2	1.05	.35	1.87	.90	.31
6 hr.	3.9	.50	.08	-	-	-

Step 12: $T_R = t/2 + T_p = 0.5/2 + .92\ (1.6) = 1.7$ hrs.

Time (hrs.)	t/T_R	q/q_p	q(cfs)
.25	.15	.04	86
.50	.29	.11	238
.75	.44	.25	540
1.00	.59	.46	992
1.25	.73	.71	1540
1.50	.88	.93	2010
1.70	1.00	1.00	2160
2.00	1.17	.83	1800
2.25	1.32	.58	1250
2.50	1.47	.42	908
3.00	1.76	.23	496
4.00	2.35	.06	130
5.00	2.94	.02	43

APPENDIX C

Sample Computations

Design of 10-year flood for Floodplain Area

Step 1: Given area = 4000 acres
Step 2: Given W = 6160 feet
Step 3: Given S = .035
Step 4: Columns 1 and 2 in table
Step 5: Fig. 13 gives column 3 in table
Step 6: Column 4 in table

Step 7: Basin lag factor $= \frac{W^{0.6}}{S^{0.4}} = \frac{(6160)^{.6}}{(.035)^{.4}} = 718$

Step 8: $t_p = .000567\ LF^{1.48} = .000567(718)^{1.48} = 9.5$ hrs.

Step 9: Column 6 in table
Step 10: Column 7 in table
Step 11: Q = XZA = (.080) (4000) = 320 cfs

Design for API = 2, June-July

(1) Time	(2) Rainfall Amount (in.)	(3) Runoff Amount (in.)	(4) X	(5) t/t_p	(6) Z	(7) XZ
15 min.	1.4	.70	2.80	.026	.017	.048
30 min.	2.0	1.08	2.16	.053	.036	.078
1 hr.	2.5	1.15	1.15	.105	.070	.080
2 hr.	3.0	1.02	.51	.210	.140	.071
3 hr.	3.2	.69	.23	.316	.200	.046
6 hr.	3.9	.12	.02	.631	.360	.007

Step 12: $T_R = t/2 + T_p = 1/2 + 1.01\ (9.5) = 10.1$ hrs.

Time (hrs.)	t/T_R	q/q_p	q(cfs)
2	.20	.07	22
4	.40	.27	86
6	.59	.57	182
8	.79	.86	275
9.0	.89	.97	310
10.1	1.00	1.00	320
12	1.19	.91	291
14	1.38	.74	236
16	1.58	.57	182
18	1.78	.46	147
20	1.97	.37	118
24	2.39	.24	77
28	2.77	.18	58
32	3.17	.12	38
36	3.57	.08	26
40	3.96	.05	16

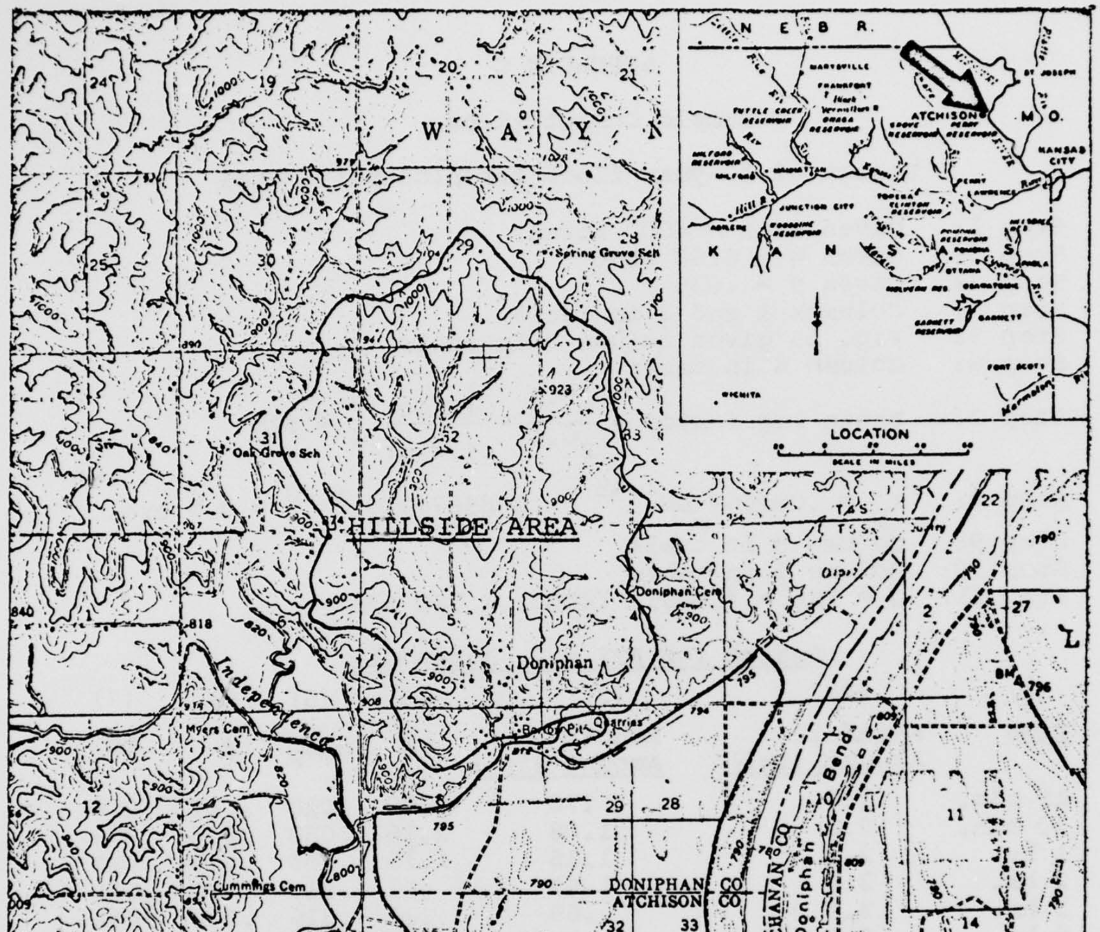


Fig. 1 Location Map of Experimental Area

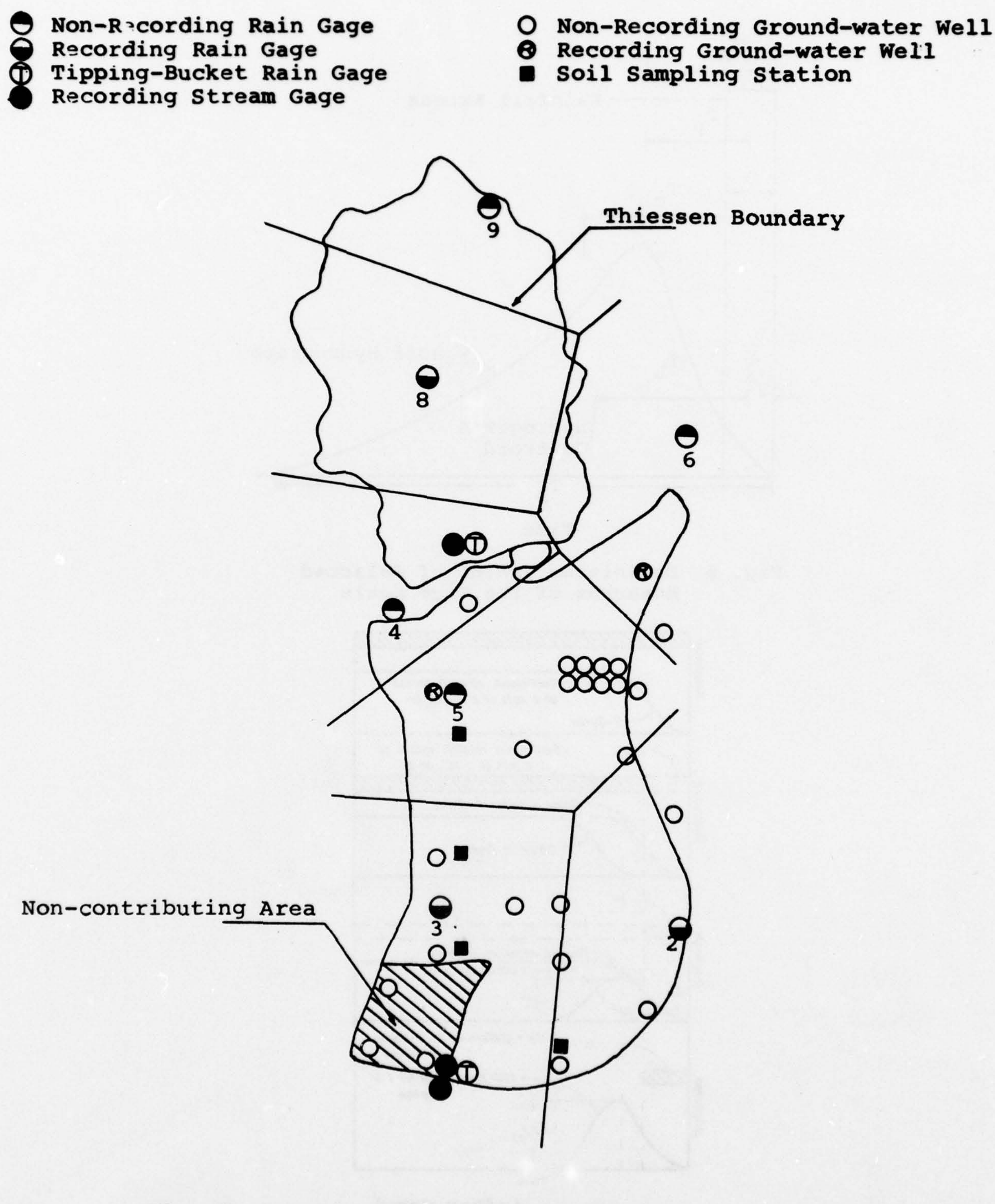


Fig. 2 Location of Basic Data Stations

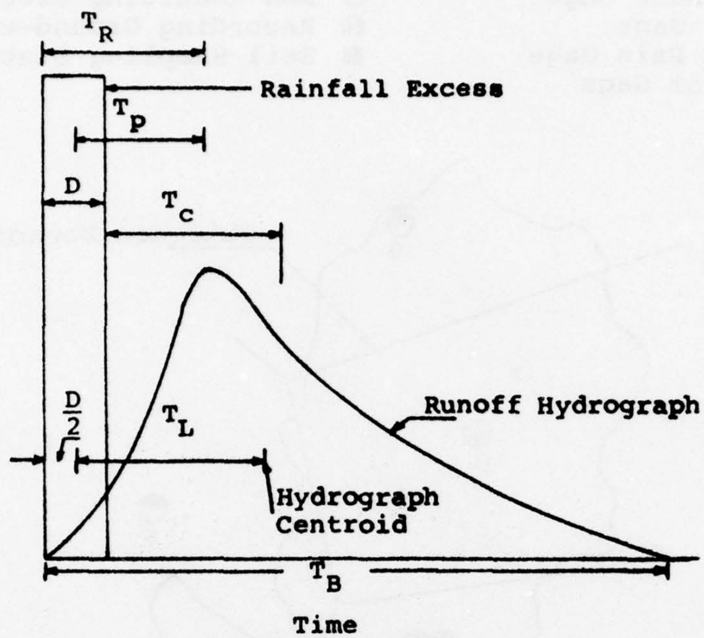
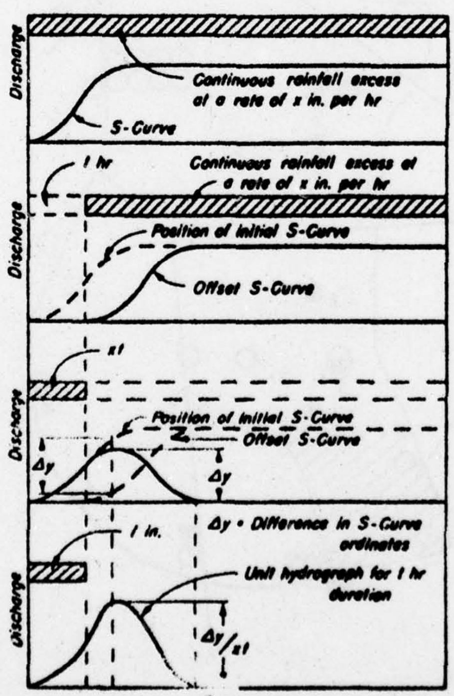


Fig. 6 Definition Sketch of Selected Measures of the Time Scale



(after Chow)

Fig. 7 General Relationship of Unitgraph and S-hydrograph

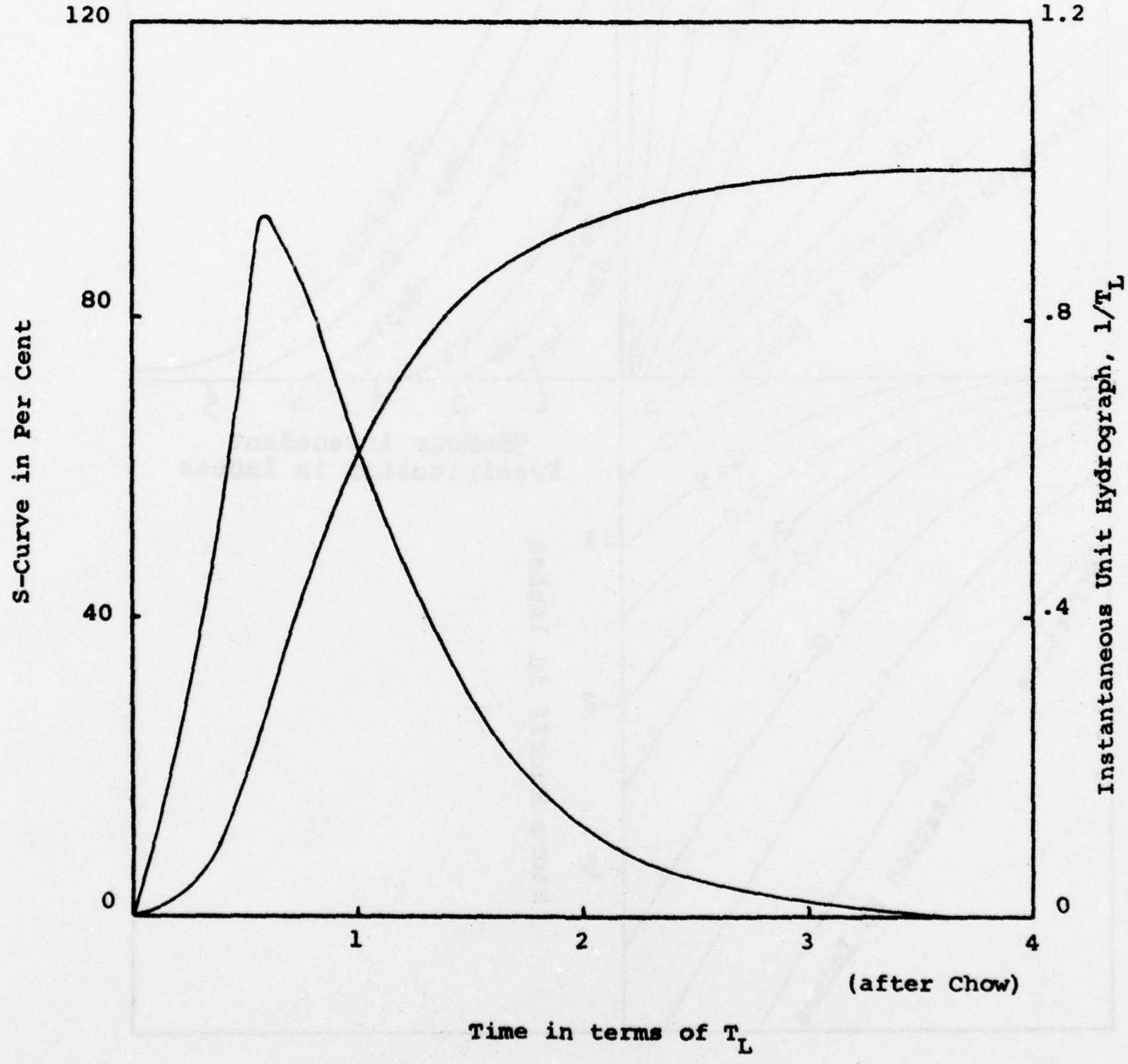


Fig. 11 Relationship of S-Hydrograph and Instantaneous Unitgraph

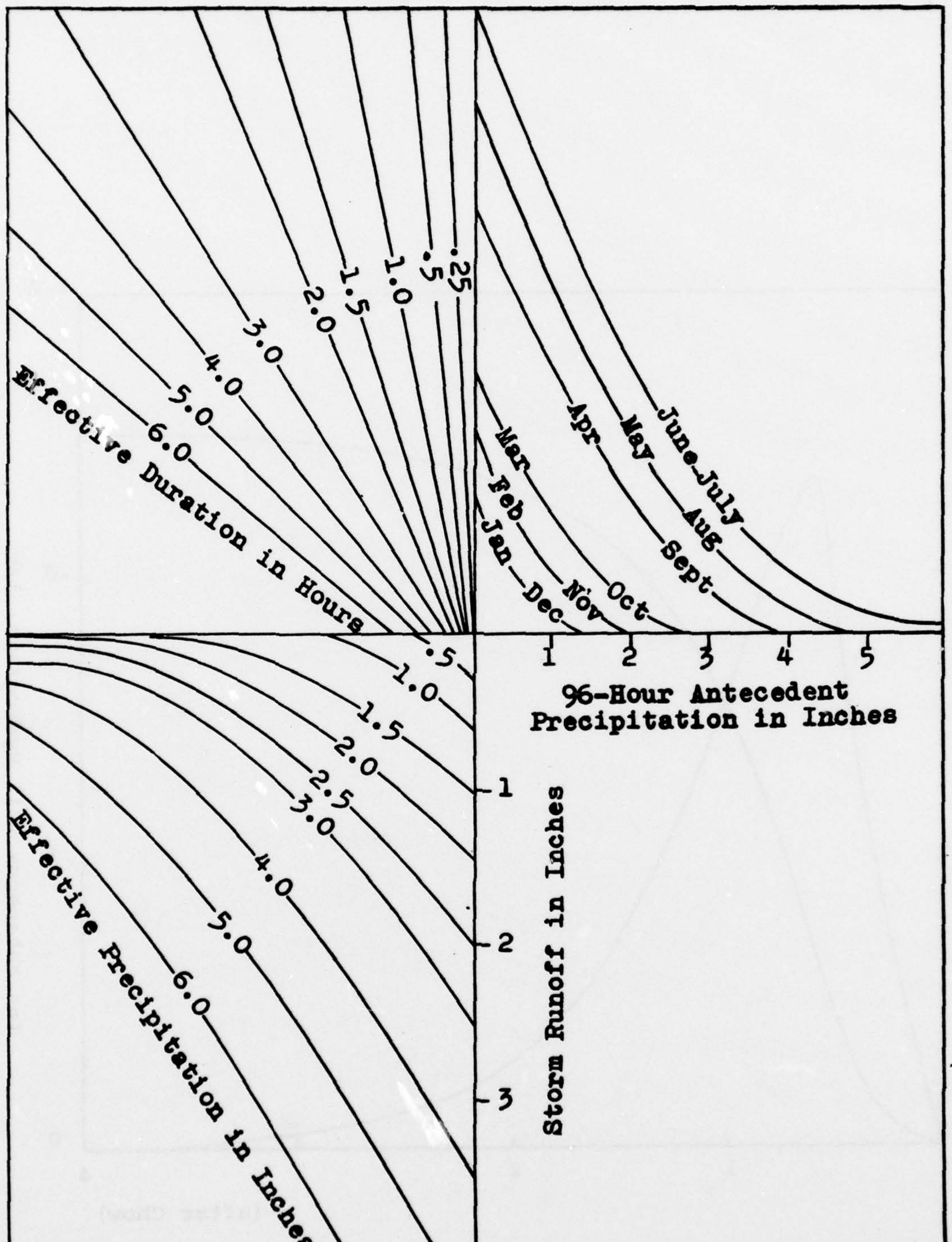


Fig. 12 Rainfall-Runoff Relationship for the Hillside

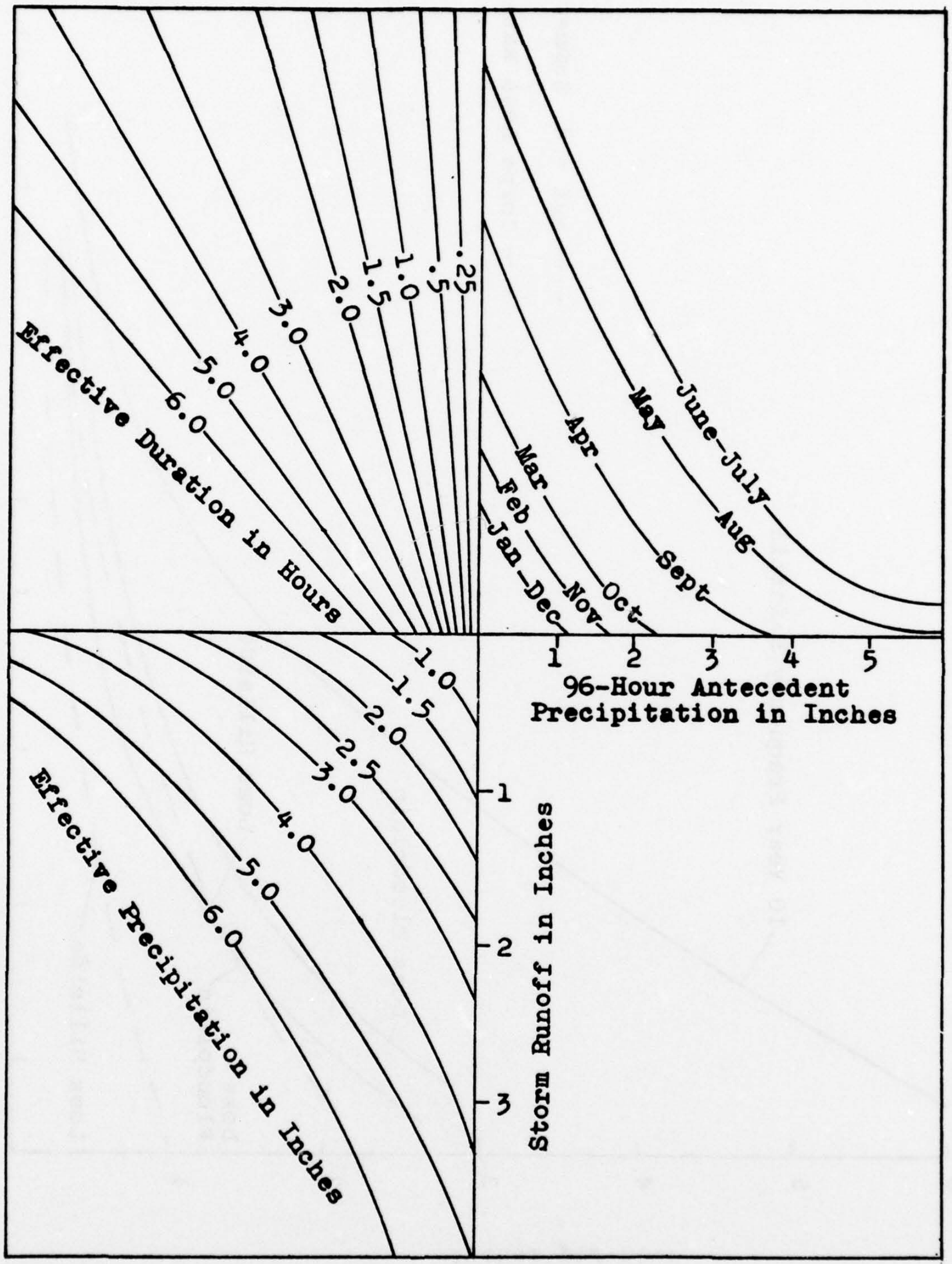


Fig. 13 Rainfall-Runoff Relationship for the Floodplain

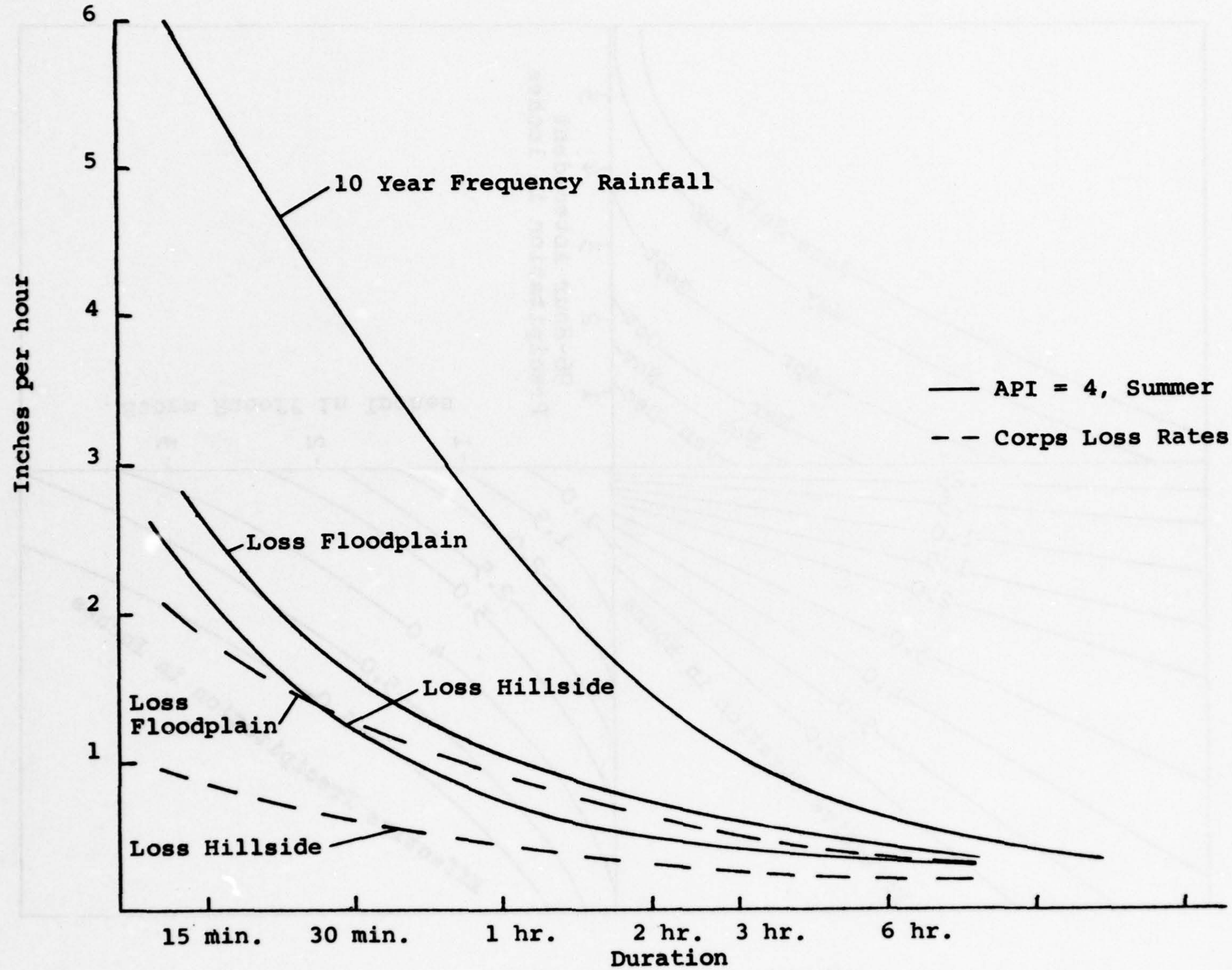


Fig. 14 Comparison of Observed Loss Rates With Current Design Values

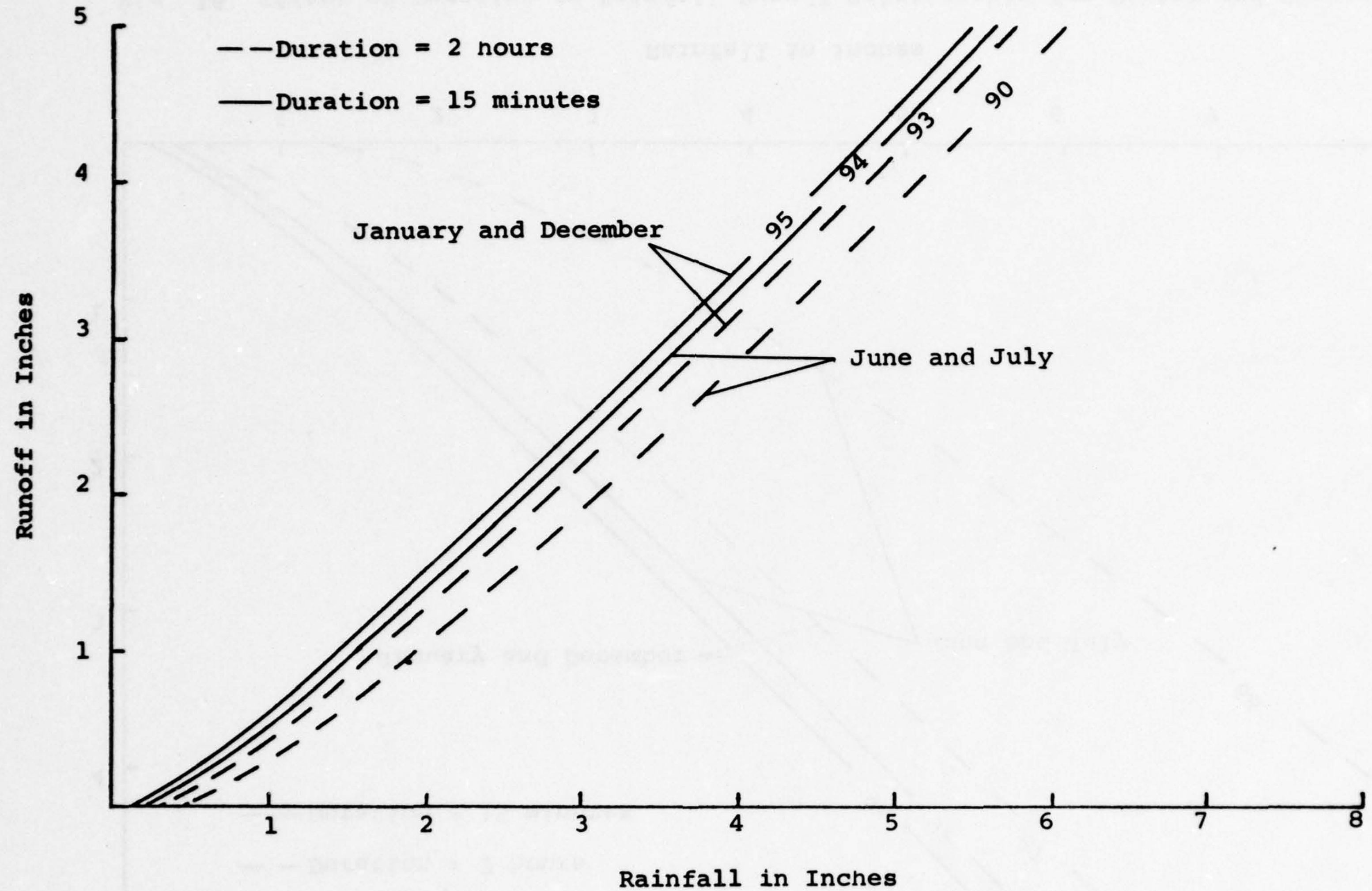


Fig. 15 Effect of Duration on Rainfall-Runoff Relationship for Winter and Summer Seasons with Antecedent Precipitation of 4 Inches, Hillside

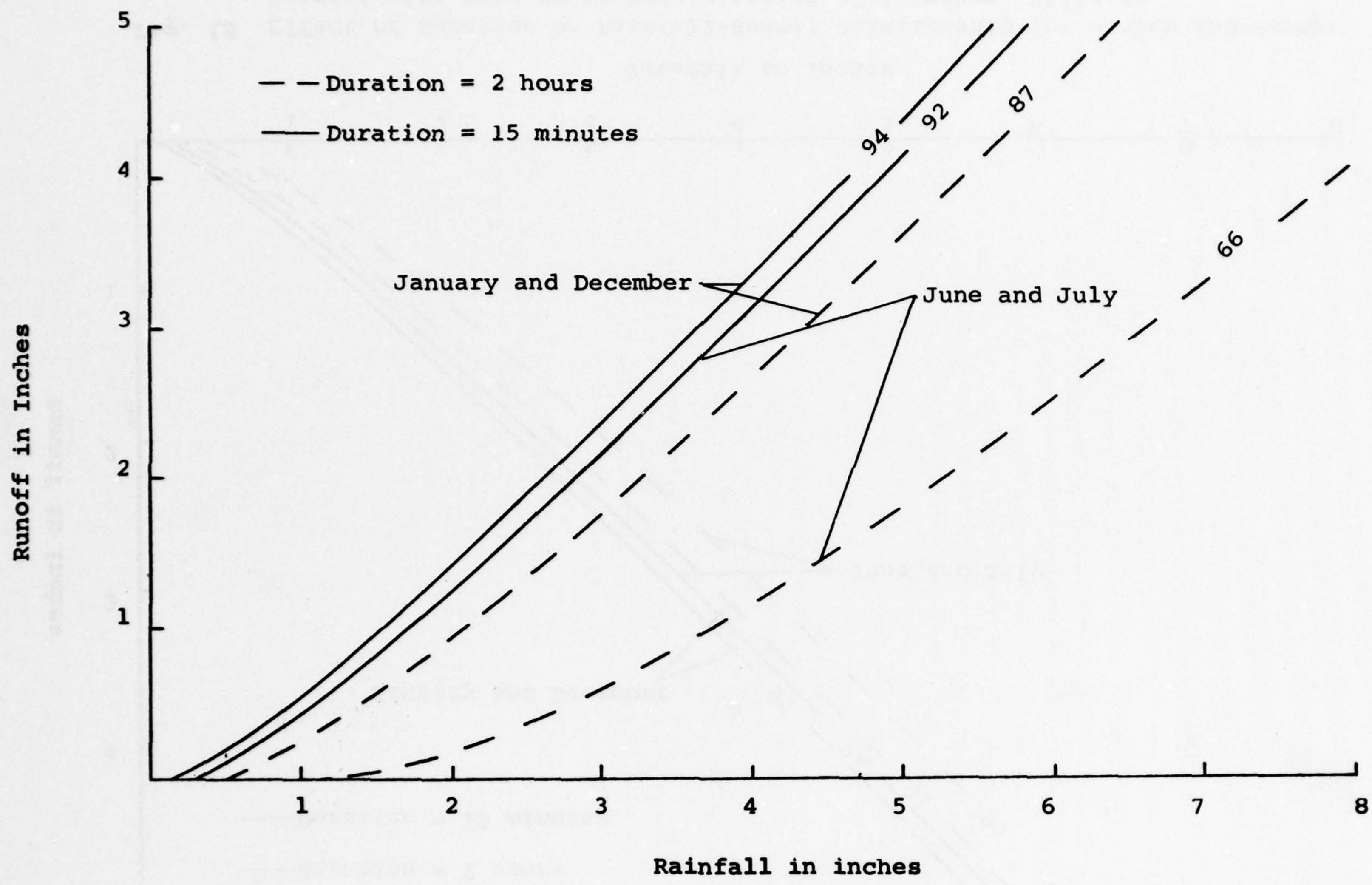


Fig. 16 Effect of Duration on Rainfall-Runoff Relationship for Winter and Summer Seasons with Antecedent Precipitation of 0 Inches

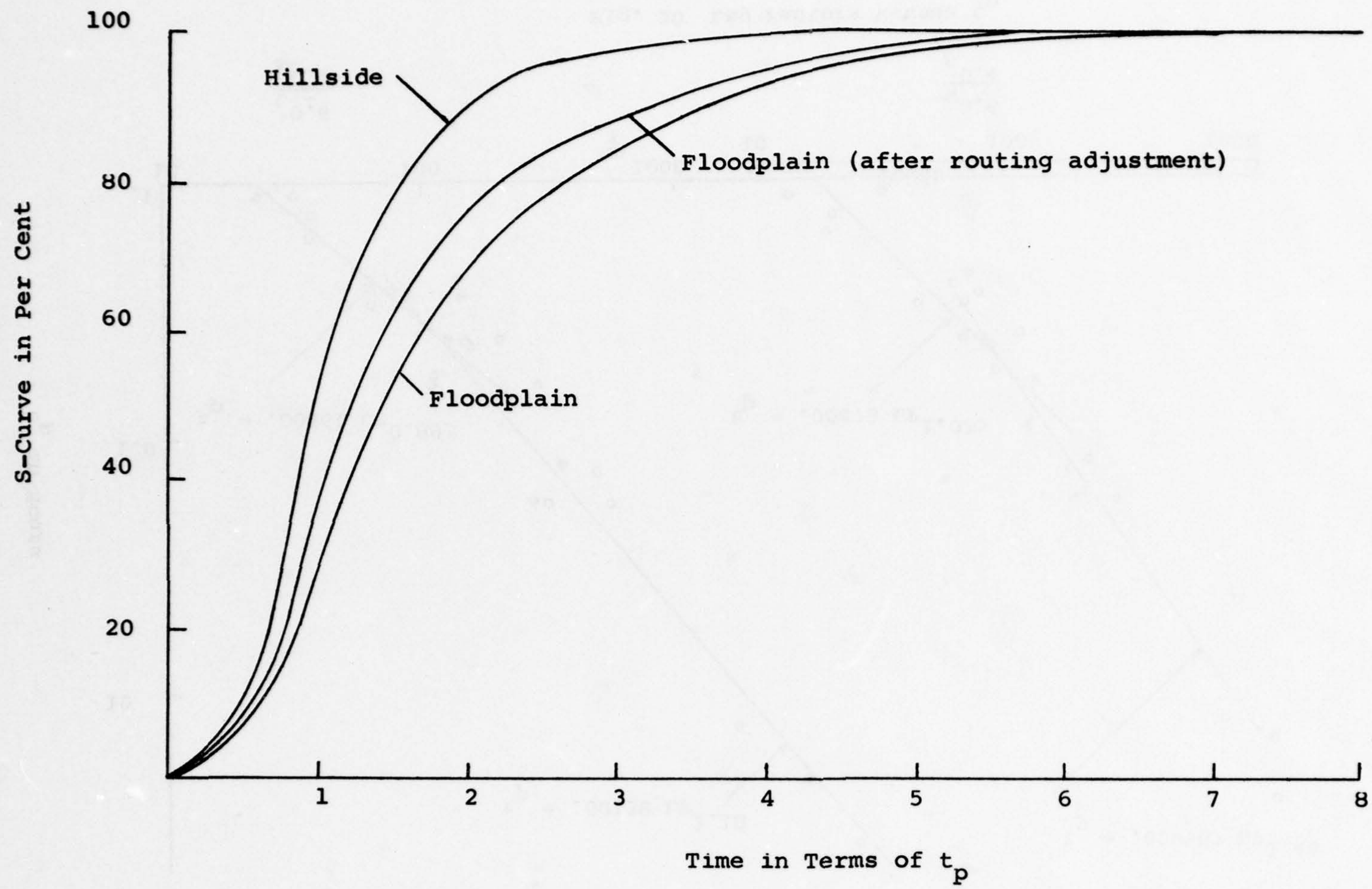


Fig. 19 Dimensionless S Hydrographs for Experimental Watersheds

Paper 7

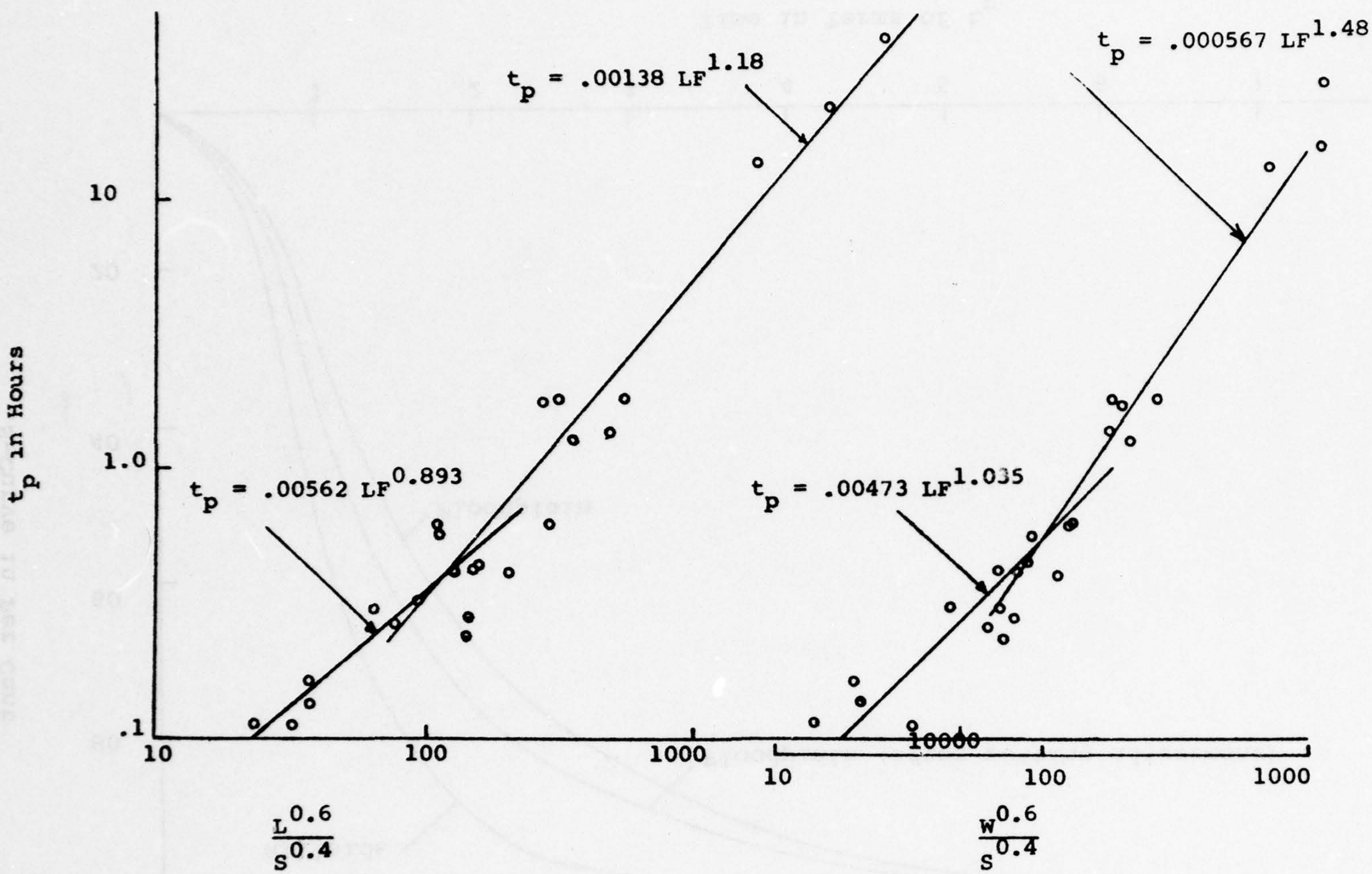


Fig. 20 Lag Factors Versus t_p

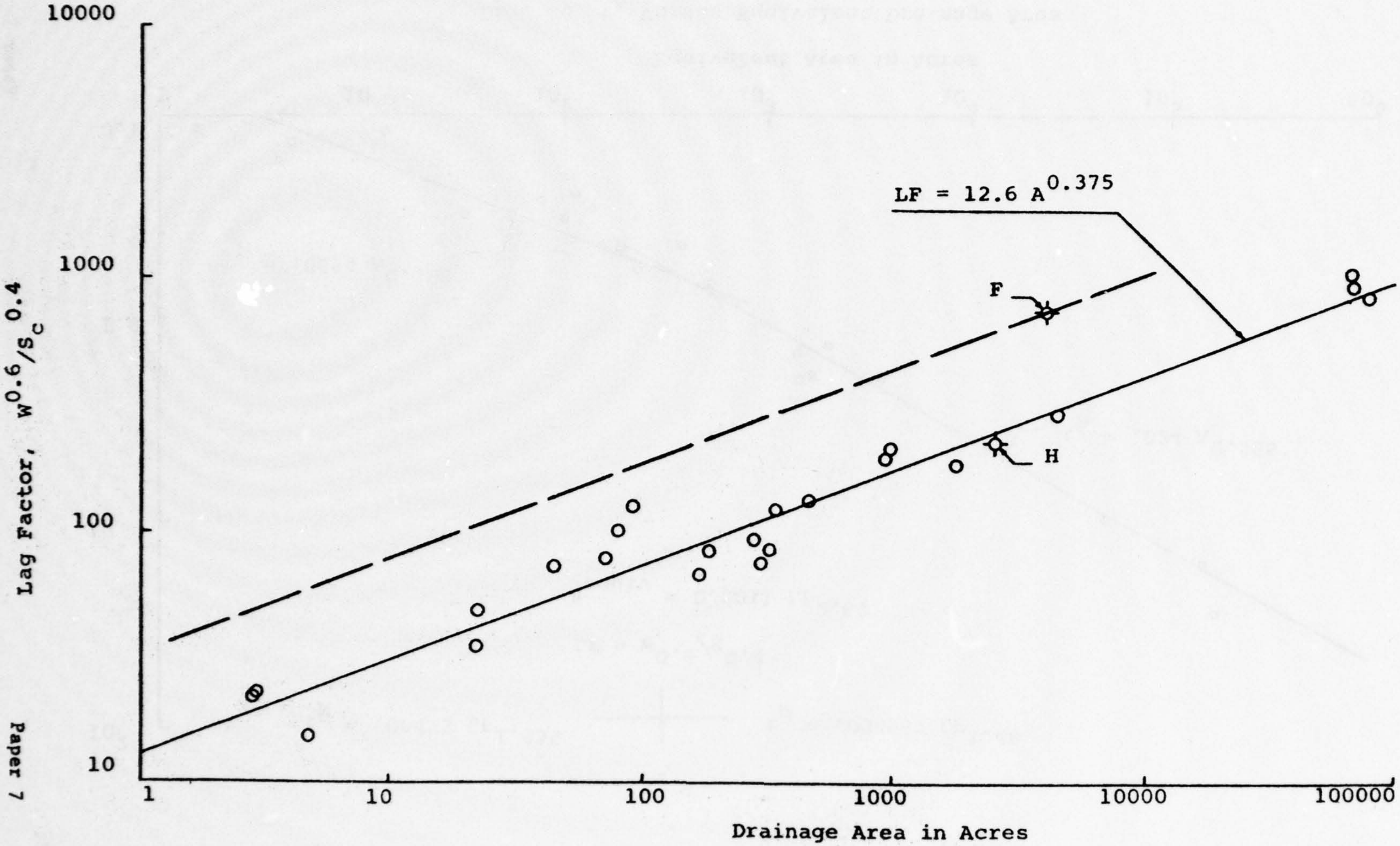


Fig. 21 Relationship of Drainage Area to Lag Factor

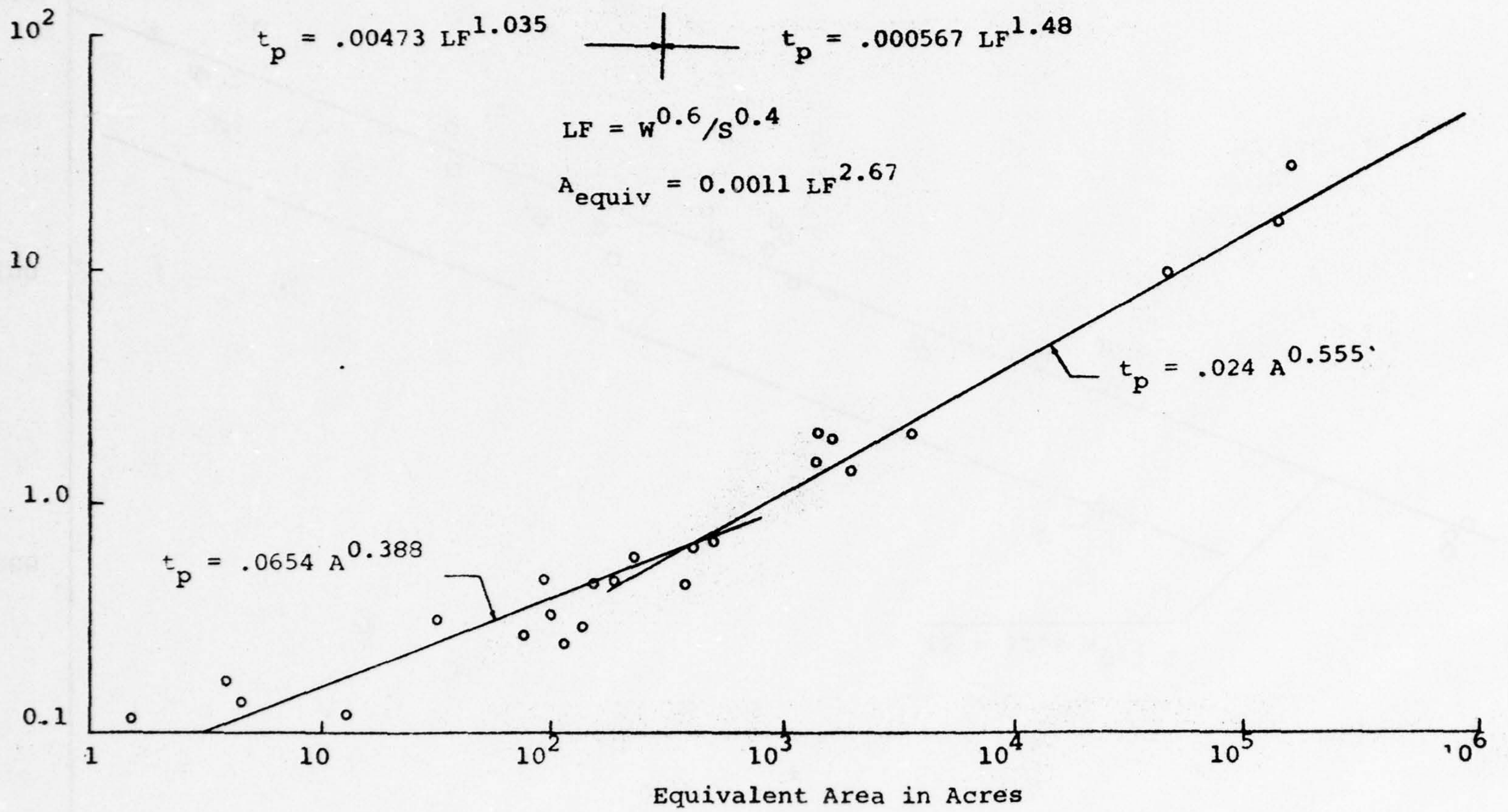


Fig. 22 t_p Versus Equivalent Drainage Area

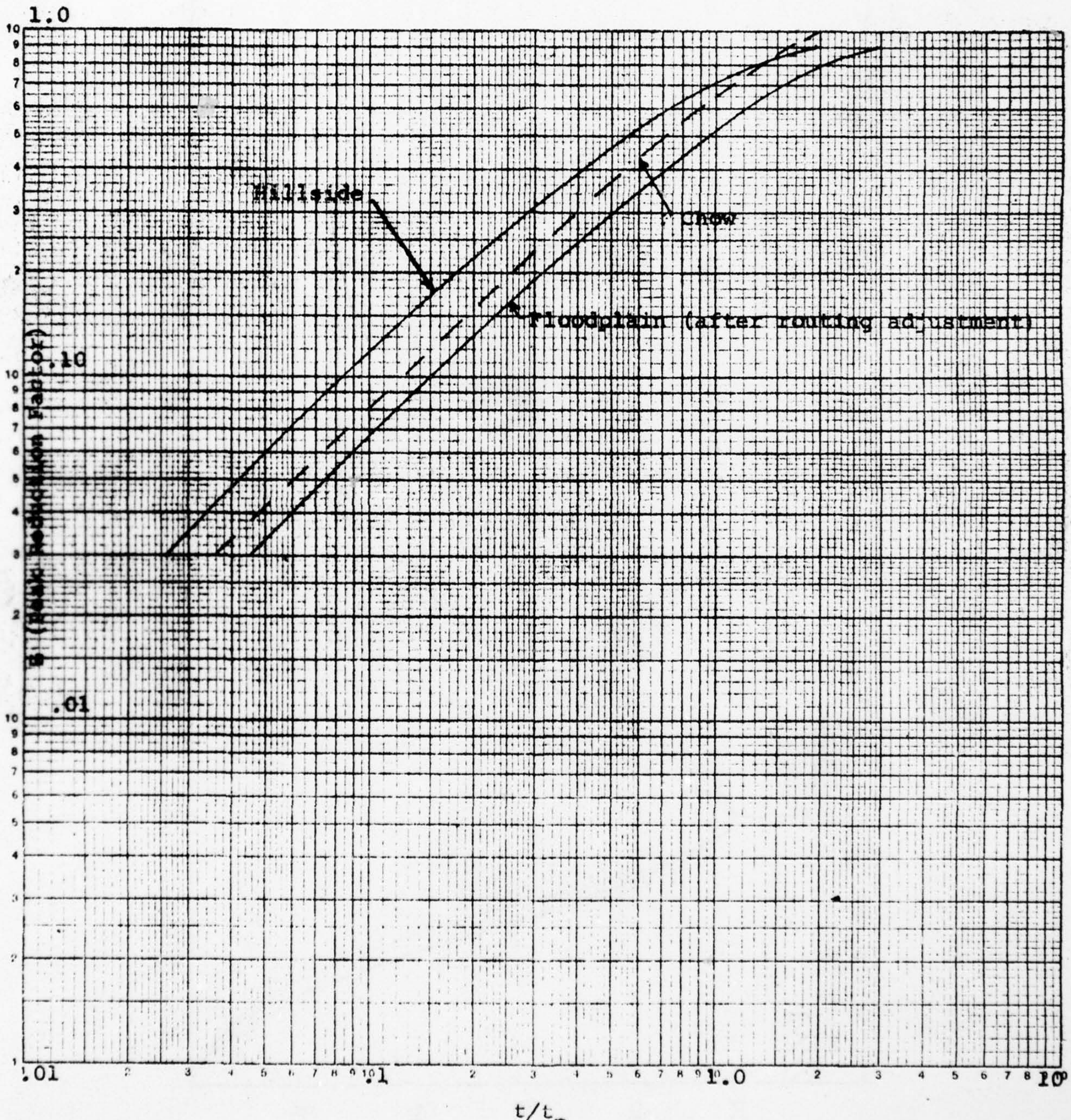


Fig. 23 ß versus t/t_p

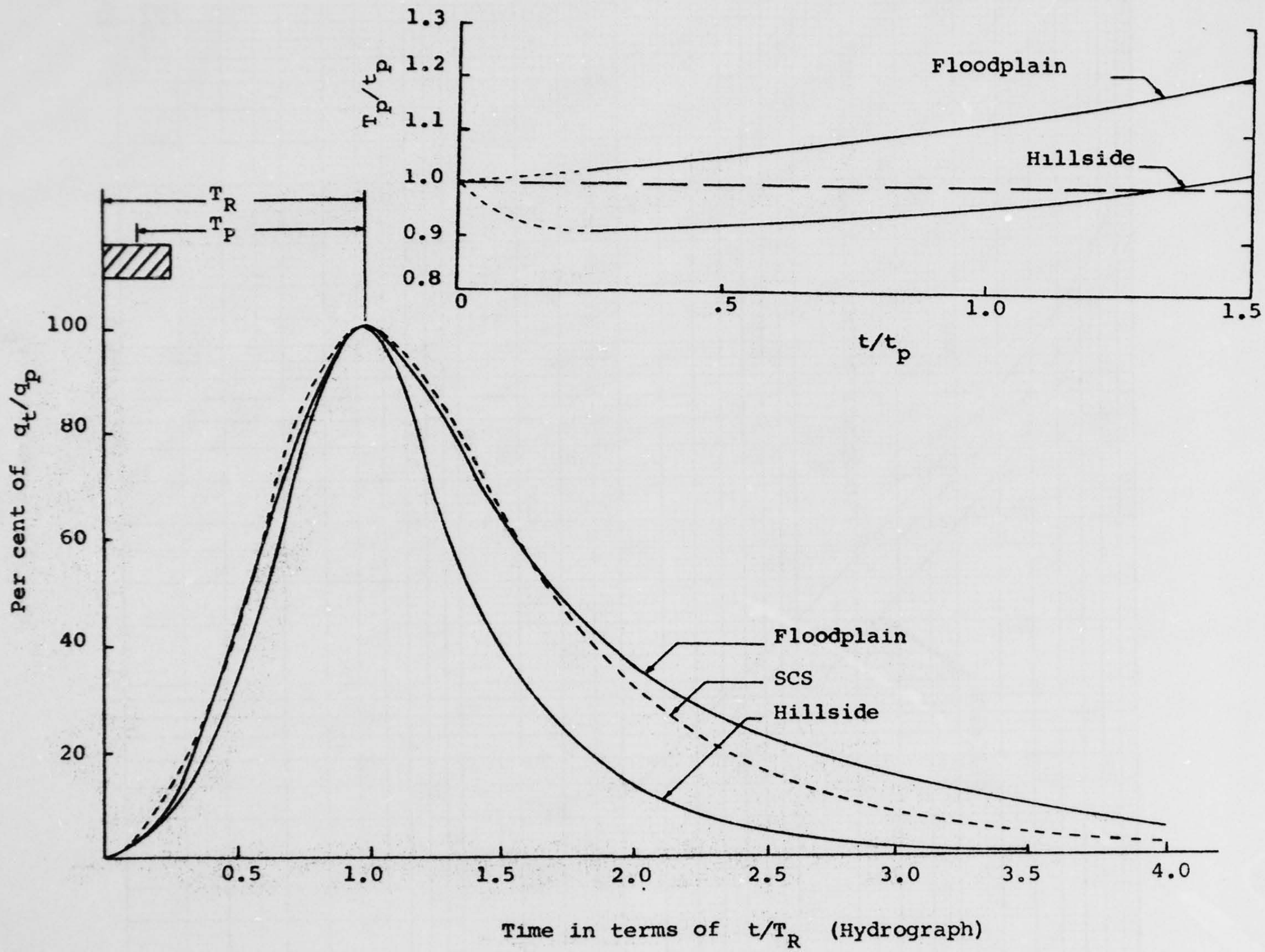


Fig. 24 Dimensionless Hydrographs

DISCUSSION OF SOME ASPECTS OF
URBAN HYDROLOGY METHODOLOGY

Discussion

Question, Mr. Beard: In our early computer applications, The Hydrologic Engineering Center used the Horner-Flynt delta-graph technique as an effective artificial device for computing a unit hydrograph from coefficients. Is this technique currently used much in the Missouri River Division?

Reply, Mr. Timberman: The method is presently used only occasionally in the Division office for special purposes and checking. However, the computer program has just been completed for a few weeks. It is believed it may be used extensively in the future as the final step in the Snyder method approach to replace manual sketching and trial volume balancing of the unit hydrograph. It is hoped to apply the Chow method of analysis described in the basic paper to urban and other hydrologic situation trial studies. It appears it may be advantageous to substitute the delta-graph procedure for the dimensionless hydrograph procedure for the actual construction of the hydrograph after time-to-peak has been determined.

The requirements of urban hydrology may throw new light on the relative desirability of various approaches. For instance, the need for adjustment of hydrograph shape based on basin factors that change with the degree of urbanization is an important consideration. It appears that delta-graph method may be especially suitable in this regard, but other methods may be equal or more suited. The Kansas City District has been using primarily Clark's method (time-area). It is expected that this will continue. However, if advantages to be gained by substitution or supplementation by the delta-graph were considered sufficient, it would not be a great problem to convert parameters to use that procedure.

URBAN HYDROLOGY IN CONNECTION WITH
CHANNEL IMPROVEMENT PROJECT AT
NEWMARKET CREEK, VIRGINIA

by Carl D. Matthias[1]

The numerous procedures involved in this study are not necessarily unique. Possibly the means of putting them together to provide hydrologic data necessary to design this channel improvement project are somewhat different.

The principal problem was the determination of flood frequencies for a proposed channel improvement with the drainage area contributing to the channel ranging from 2.33 square miles to 8.54 square miles with various conditions of watershed development and various portions of channel improved. This and the need to consider present and probable future conditions of watershed development indicated a large number of combinations for which flood flow frequency determinations were required.

The watershed and channel studied was Newmarket Creek, Virginia. Exhibit 1 indicates this area. The principal source of damage was the Newmarket Shopping Center. This can be recognized as a positive example of unwise use of the flood plain, particularly by those of you who have ever been involved in the flood plain management program. A large shopping center, similar to the thousands of such centers being constructed throughout the country, was constructed directly on the flood plain. A channel, which proved to be inadequate was provided around the end of the shopping center. to carry the flow of the stream.

It is another example of the tremendous increase in flows which result after substantial development takes place on the watershed. This may be exaggerated by the fact that previous flooding may have gone unnoticed or caused no concern where there was nothing of consequence to be flooded. Nevertheless, it is obvious that all areas in which significant development takes place sustain a corresponding substantial increase in the frequency of flooding.

[1] Chief, Hydrology Section, Norfolk District

As always, the event which triggered this study was occurrence of a damaging flood. In August 1960 3-1/2 to 4 inches of rain were estimated to have fallen on the watershed in about 7 hours. Flooding occurred along the creek above U. S. 258, flowed over the road for a considerable length, covered the parking area to the shopping center and was within inches of flooding stores in the shopping center. A flood nearly as severe occurred the following month.

Slopes in the main study area are relatively flat, about 0.5 feet per 1000 feet. Also, the cross section of the flood plain on either side is flat so that, under natural conditions, considerable water is stored along the stream and runoff is slow. You may note that the lower portion of the stream is in Tidewater. No plan of improvement was considered in this area. There were many variables as previously mentioned such as differing drainage areas, possibility of improvement of varying portions of channel, and need to consider present and anticipated future conditions. The study was developed in such a way that these variables could be easily considered in determination of flood flows and that these flows would be consistent throughout the watershed.

As is the usual case when determining flood frequency in small areas, there were no streamflow records available on this or any other stream which could be considered reasonably characteristic of the area.

Unit hydrographs were derived using some adjustment of the methodology of the U. S. Soil Conservation Service as contained in its National Engineering Handbook, Section 4, Hydrology. Pertinent relationships are shown on exhibit 2. The principal elements of data needed to define the unit hydrograph consist of the peak ($q = 484/T_p$), the time from beginning of rainfall excess to the peak, and the shape as determined from the dimensionless unit hydrograph presented in the SCS handbook noted previously and shown in the following table.

Table 1. RATIOS FOR BASIC DIMENSIONLESS HYDROGRAPH AND MASS CURVE
(SCS table 3.16-1)

Time ratios (T/T_p)	Hydrograph discharge ratios (q/q_p)	Mass curve ratios (Q_a/Q)
0	0	0
0.1	0.015	0.001
.2	.075	.006
.3	.16	.018
.4	.28	.037
.5	.43	.068
.6	.60	.110
.7	.77	.163
.8	.89	.223
.9	.97	.300
1.0	1.00	.375
1.1	.98	.450
1.2	.92	.517
1.3	.84	.577
1.4	.75	.634
1.5	.66	.683
1.6	.56	.727
1.8	.42	.796
2.0	.32	.848
2.2	.24	.888
2.4	.18	.916
2.6	.13	.939
2.8	.098	.954
3.0	.075	.967
3.5	.036	.984
4.0	.018	.993
4.5	.009	.997
5.0	.004	.999
infinity	0	1.000

You will note that a lag time, $L = 0.8\ T_c$ was used in the study instead the $0.6\ T_c$ indicated by the SCS as being an average. In view of the flatness of the basin the L was increased somewhat to $0.8\ T_c$ which has the effect of broadening the unit hydrograph and lowering the peak.

The shape of the watershed was generally rectangular and the width was relatively narrow. The assumption was made that, for a given time of concentration, the time to peak was the same for any watershed area and the ordinates of the unit hydrograph were in direct proportion to the watershed area. Some arbitrary adjustment was made in ordinates, other than the peak ordinate, so that they added up to one watershed inch or 645 cfs-hours.

Using procedures previously described, unit hydrographs were derived for a drainage area of one square mile for a number of different times of concentration covering the required range expected in the study with the results listed on the following table.

Table 2. UNIT HYDROGRAPHS APPLICABLE TO NEWMARKET CREEK

Time in hours	Discharge in c.f.s. resulting from 1 inch rainfall excess in 1 hour over 1 square mile for T_c (time of concentration) in hours indicated				
	0.6	1.9	4.4	6.9	9.3
0	0	0	0	0	0
1	484	105	14	3	1
2	124	242	53	16	7
3	30	156	100	35	15
4	8	75	121	57	27
5	0	36	104	73	39
6		18	79	81	50
7		8	54	75	55
8		4	37	64	60
9		2	26	52	56
10		0	18	41	52
11			13	32	46
12			9	25	40
13			6	19	33
14			4	16	27
15			3	13	22
16			2	10	19
17			1	7	16
18			1	6	14
19			1	5	11
20			0	4 (a)	9 (a)

(a) Entire falling leg not shown.

Precipitation frequency applicable to the area under study was obtained from U. S. Weather Bureau Technical Paper No. 29, Part 3, The Middle Atlantic Region "Rainfall Intensity - Frequency Regime." The watershed areas studied ranged from 2.3 to 8.5 square miles and times of concentration from 2 to 9 hours. Storm rainfall was developed by conventional methods for a 12-hour period for 2, 10, and 100-year events applicable to a watershed area of 5 square miles. There is little reduction in rainfall from point rainfall so that this is not particularly significant.

The 2, 10, and 100-year peak flood discharges were obtained by application of appropriate storm rainfall to the unit hydrographs listed in table 2. Rainfall losses of 1.2 inches initially plus uniform loss of 0.2 inch per hour were used to represent present conditions and losses of 0.8 inch initially plus 0.1 inch per hour were used to represent probable future conditions with the watershed fully developed. Shown on the following table are peak discharges for a one square mile area determined as previously described. Also shown are discharges for a five square mile area which, as previously indicated, would be in direct proportion to that for one square mile. Use of a five square mile area to determine discharges provided a convenient point for plotting curves as described in paragraphs which follow.

Table 3. PEAK FLOOD DISCHARGES IN CFS FOR FREQUENCY AND STAGE OF DEVELOPMENT INDICATED

T_c	Present			Fully Developed		
	2 yr.	10 yr.	100 yr.	2 yr.	10 yr.	100 yr.
Discharges for 1 sq. mi. watershed area						
0.6	474	850	1360	634	980	1480
1.9	240	450	780	338	550	860
4.4	122	240	430	180	310	500
6.9	82	164	-	125	216	-
9.3	61	121	220	92	162	270
Discharges for 5 sq. mi. watershed area						
0.6	2370	4250	6800	3170	4900	7400
1.9	1200	2250	3900	1690	2750	4300
4.4	610	1200	2150	900	1500	2500
6.9	410	820	-	625	1080	-
9.3	305	605	1100	460	810	1350

A curve of T_c vs. discharge for a 5 square mile area for each of the flood frequencies, both for present and fully-developed conditions were plotted and discharges for even values of time of concentration were interpolated therefrom. These discharges were plotted on a graph and a straight line drawn between the origin and each of the times of concentration to form a family of curves. This is based on the assumption that for a given time of concentration, the peak flood discharge, as well as peak of the unit hydrograph, are directly proportional to the drainage area. Such a set of curves for present conditions in the watershed for the 100, 10, and 2-year floods are shown on Exhibits 3, 4, and 5, respectively and for a fully developed condition are shown on Exhibits 6, 7, and 8 respectively.

With these curves it is possible to determine the peak discharges from a small drainage area for any given time of concentration. Discharges for any location and condition were determined by entering the curves with the approximate drainage area and time of concentration. The time of concentration was estimated from that under existing conditions and adjusted by the change in time of travel in the channel which would obtain if a portion of the channel was improved as assumed in the plan of improvement being considered.

An estimate of the time of concentration at U. S. 258 under natural conditions was obtained from a consideration of the rainfall record and inquiries of local inhabitants as to the time of the flood crest in the August 1960 flood. Estimates for all other conditions and locations were based on probable channel velocity, length of channel, extent of improvement studied, and judgement. Discharges for various floods at different locations and under varying conditions are shown in the following table.

Table 4.

DISCHARGE FREQUENCY DATA, NEWMARKET CREEK, VA.

Location	Drainage area in sq. mi.	Time of concentration and discharge for recurrence interval indicated					
		2-year		10-year		100-year	
		Tc, hrs	Q, cfs	Tc, hrs	Q, cfs	Tc, hrs	Q, cfs
PRESENT WATERSHED DEVELOPMENT - EXISTING CHANNEL							
Dresden Drive	2.33	5.0	260	5.5	500	6.0	800
Bellwood Road	3.36	5.5	340	6.0	650	6.5	1,000
Todds Lane	4.90	6.0	460	6.5	870	7.0	1,400
Upper end of Government Ditch (a)	5.32	6.25	480	6.75	920	7.25	1,470
U. S. 258 (natural)	6.39	7.0	520	7.5	1,000	8.0	1,600
U. S. 258 (b)	6.39	7.0	360	7.5	800	8.0	1,370
Orcutt Avenue (natural)	6.82	7.25	530	7.75	1,025	8.25	1,660
Orcutt Avenue (b)	6.82	7.25	380	7.75	825	8.25	1,430
Chestnut Avenue (natural)	7.75	7.75	570	8.25	1,080	8.75	1,750
Chestnut Avenue (b)	7.75	7.75	460	8.25	880	8.75	1,520
Corp. limit (natural)	8.54	8.0	600	8.5	1,150	9.0	1,870
Corp. limit (b)	8.54	8.0	510	8.5	960	9.0	1,650
Orcutt Avenue (c)	0.43	2.00	140	2.00	240	2.00	350
Chestnut Avenue (c)	1.36	2.75	360	2.75	610	2.75	970
Corp. limit (c)	2.15	3.25	500	3.25	860	3.25	1,350
PRESENT WATERSHED DEVELOPMENT - IMPROVED CHANNEL TO DRESDEN DRIVE							
Dresden Drive	2.33	5.0	260	5.5	500	6.0	800
Bellwood Road	3.36	5.35	350	5.85	680	6.35	1,050
Todds Lane	4.90	5.50	490	6.00	940	6.50	1,500
Upper end of Government Ditch (a)	5.32	5.75	520	6.25	980	6.75	1,600
Upper end of Government Ditch (d)	6.39	5.75	620	6.25	1,180	6.75	1,900
James River (d)	6.88	6.25	620	6.75	1,180	7.25	1,900
James River (e)	5.81	6.25	520	6.75	1,000	7.25	1,600
WATERSHED FULLY DEVELOPED - CHANNEL IMPROVED TO HARPERSVILLE ROAD							
Dresden Drive	2.33	3.75	490	3.75	830	3.75	1,320
Bellwood Road	3.36	4.75	570	4.75	990	4.75	1,600
Todds Lane	4.90	5.00	790	5.00	1,380	5.00	2,220
Upper end of Government Ditch (a)	5.32	5.25	840	5.25	1,450	5.25	2,350
Upper end of Government Ditch (d)	6.39	5.75	930	5.75	1,610	5.75	2,640
James River (d)	6.88	6.25	930	6.25	1,630	6.25	2,660
James River (e)	5.81	6.25	790	6.25	1,380	6.25	2,240
WATERSHED FULLY DEVELOPED - EXISTING CHANNEL							
Dresden Drive	2.33	5.0	380	5.5	630	6.0	930
Bellwood Road	3.36	5.5	500	6.0	830	6.5	1,260
Todds Lane	4.90	6.0	680	6.5	1,130	7.0	1,680
Upper end of Government Ditch (a)	5.32	6.25	720	6.75	1,180	7.25	1,800
U. S. 258	6.39	7.0	780	7.5	1,280	8.0	1,970
U. S. 258 (b)	6.39	7.0	600	7.5	1,060	8.0	1,720
Orcutt Avenue (natural)	6.82	7.25	810	7.75	1,330	8.25	2,070
Orcutt Avenue (b)	6.82	7.25	630	7.75	1,110	8.25	1,820
Chestnut Avenue (natural)	7.75	7.75	870	8.25	1,400	8.75	2,220
Chestnut Avenue (b)	7.75	7.75	690	8.25	1,190	8.75	1,970
Corp. limit (natural)	8.54	8.0	930	8.5	1,500	9.0	2,370
Corp. limit (b)	8.54	8.0	760	8.5	1,290	9.0	2,130

(a) Excluding tributary on left bank at beginning of Government Ditch.
(b) As modified by present diversion by Government Ditch.
(c) Present conditions and future conditions assumed to be the same as area is now approximately 100 percent developed.
(d) Assuming all water above U. S. 258 diverted to James River.
(e) Assuming no water from tributary on left bank or between Government Ditch and U. S. 258 diverted.

The discharge data developed compared favorably with data obtained by use of a procedure developed by Franklin F. Snyder, F. ASCE (Proceedings paper 1808) Journal of the Hydraulics Division, October 1958.

Construction of this project has been completed as follows:

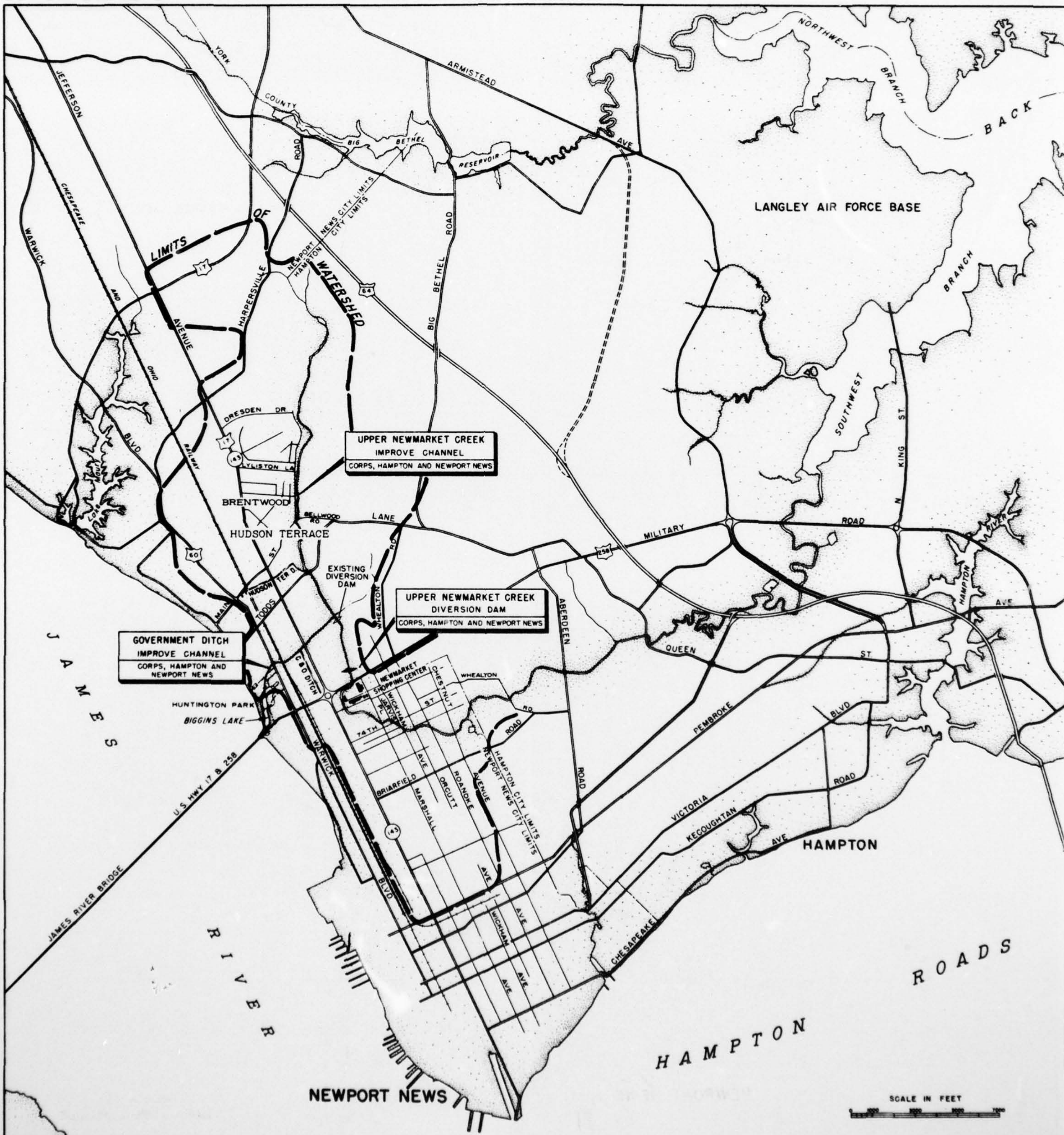
a. Channel improved from U. S. 258 to Dresden Drive as shown on map to carry the 25-year future flood.

b. Diversion dam at U. S. 258 and excavation of Government Ditch to divert water to the James River.

They are now asking us to study the possibility of improving the channel above Dresden Drive where the drainage area decreases below 2.3 square miles.

Norfolk District, Corps of Engineers
Norfolk, Va. 23510
18 Aug 70

CORPS OF ENGINEERS



U. S. ARMY

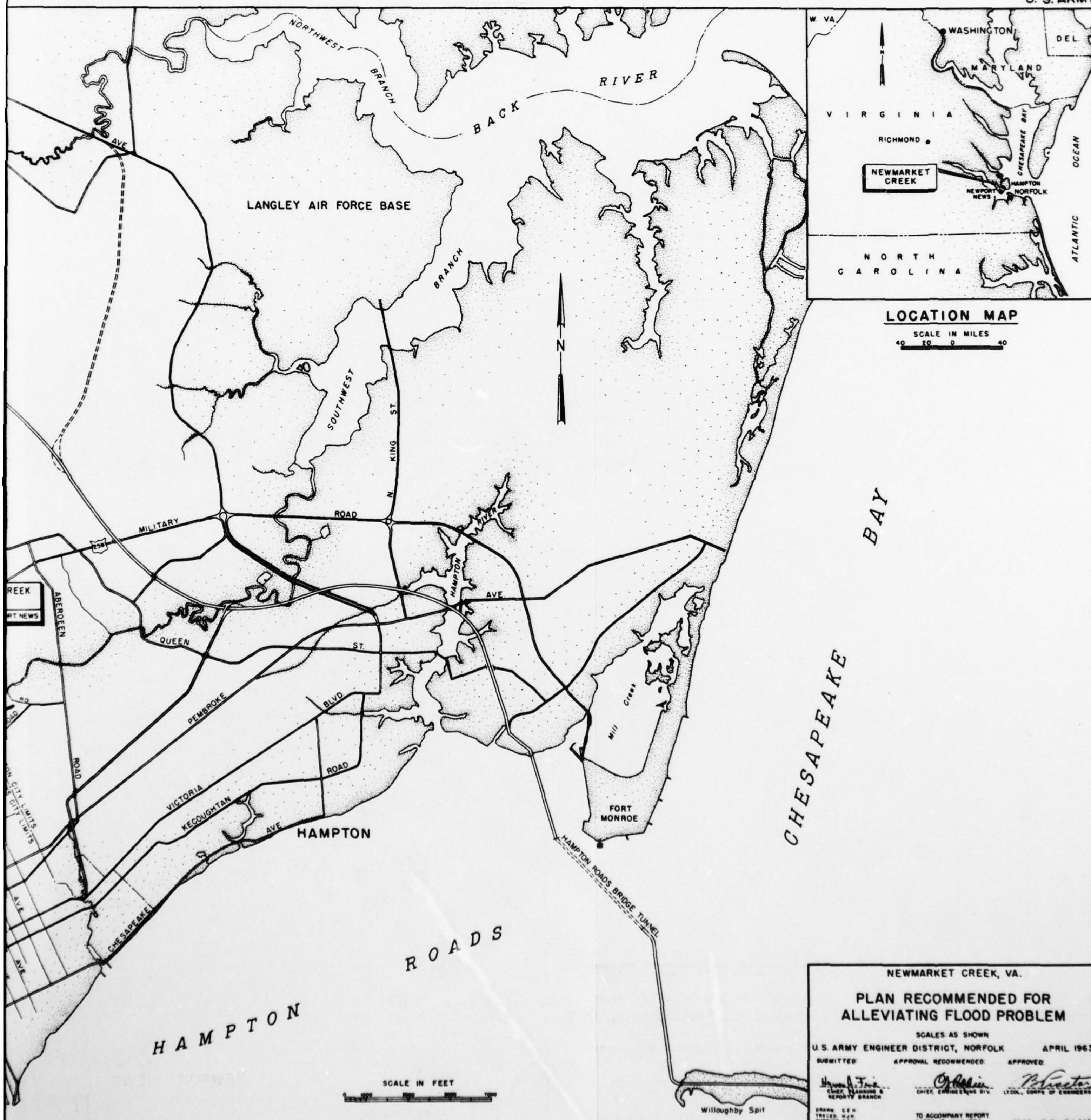
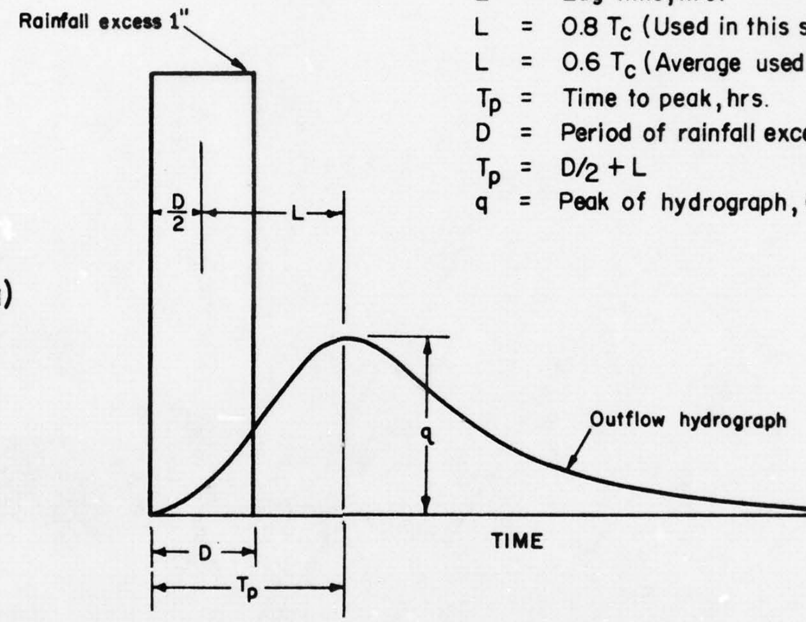


Exhibit 1

2

T_c = Time of concentration, hrs.
L = Lag time, hrs.
L = 0.8 T_c (Used in this study)
L = 0.6 T_c (Average used by SCS)
T_p = Time to peak, hrs.
D = Period of rainfall excess = 1 hr.
T_p = $D/2 + L$
q = Peak of hydrograph, CFS

(a)



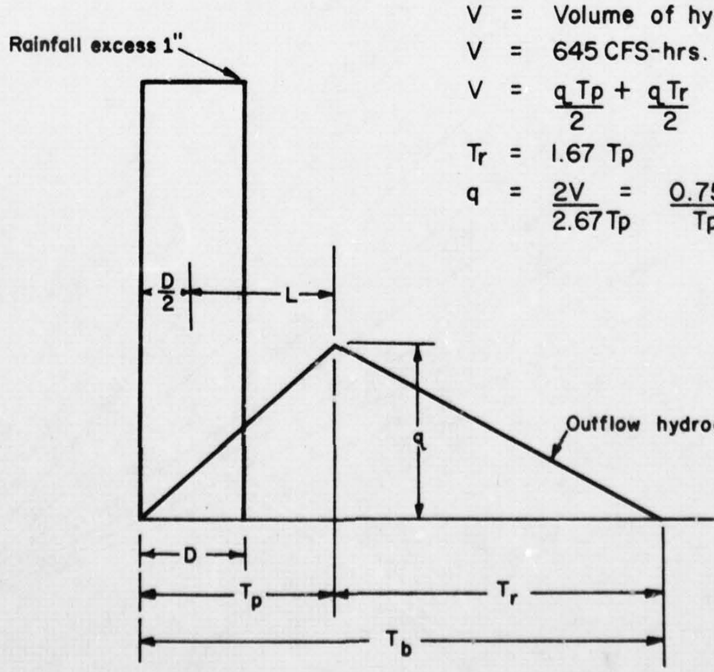
V = Volume of hydrograph
V = 645 CFS-hrs. for 1" over 1 sq. mi.

$$V = \frac{q\,Tp}{2} + \frac{q\,Tr}{2}$$

$$T_r = 1.67\ Tp$$

$$q = \frac{2V}{2.67\,Tp} = \frac{0.75V}{Tp} = \frac{484}{Tp}$$

(b)



From SCS figure 3.16-1

AD-A052 602 HYDROLOGIC ENGINEERING CENTER DAVIS CALIF F/G 8/8
PROCEEDINGS OF A SEMINAR ON URBAN HYDROLOGY HELD ON 1-3 SEPTEMB--ETC(U)
SEP 70 E F CHILDS, G S HARE, D L ROBEY
UNCLASSIFIED HEC-SP-03 NL

3 OF 4
AD
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05260

1.0 4.5 2.8 2.5
5.0 3.2 2.2
5.6 3.6
6.3 4.0 2.0
1.1
1.8
1.25 1.4 1.6

MICROCOPY RESOLUTION TEST CHART
NATIONAL BUREAU OF STANDARDS-1963-A

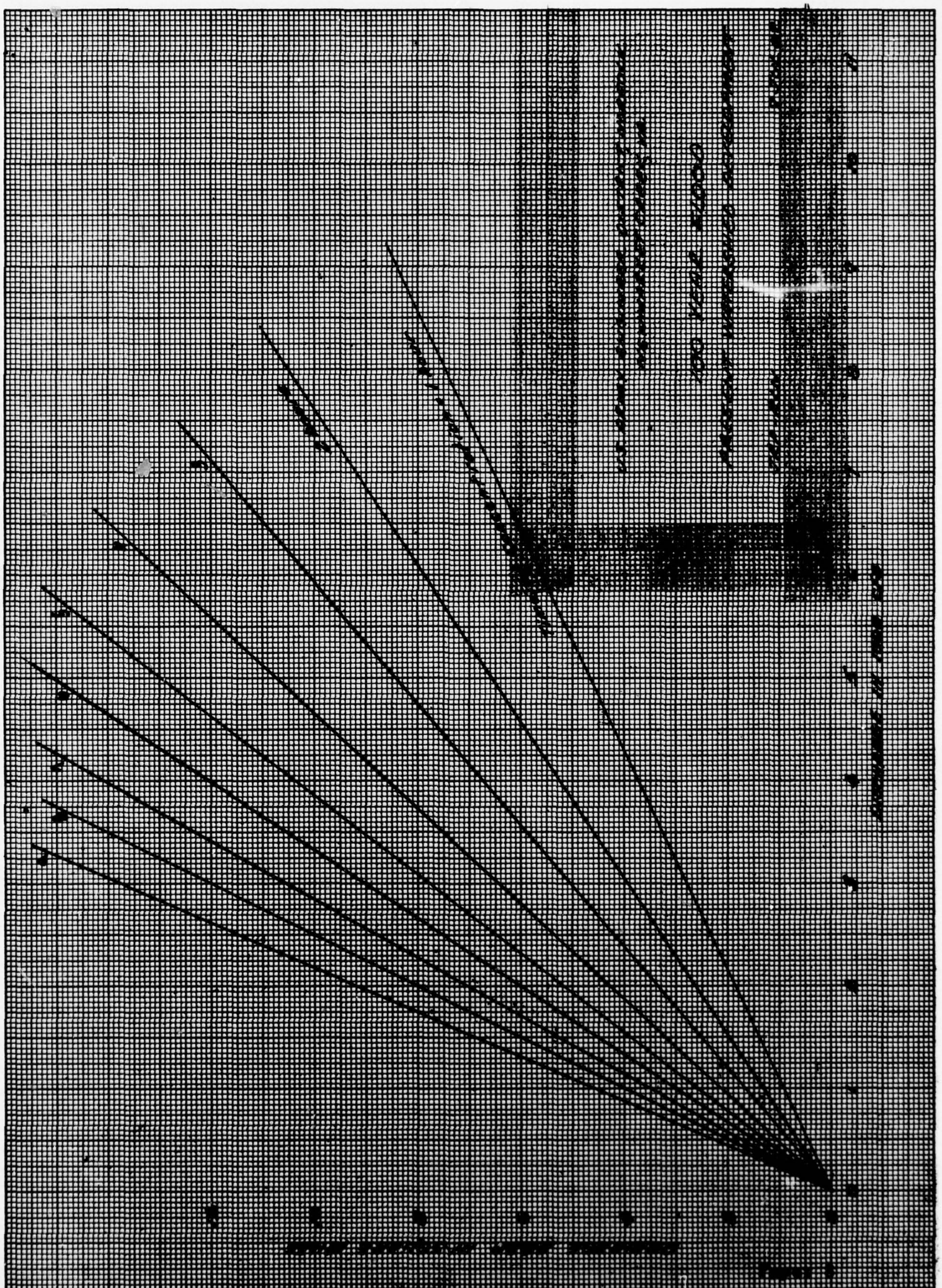


Exhibit 3

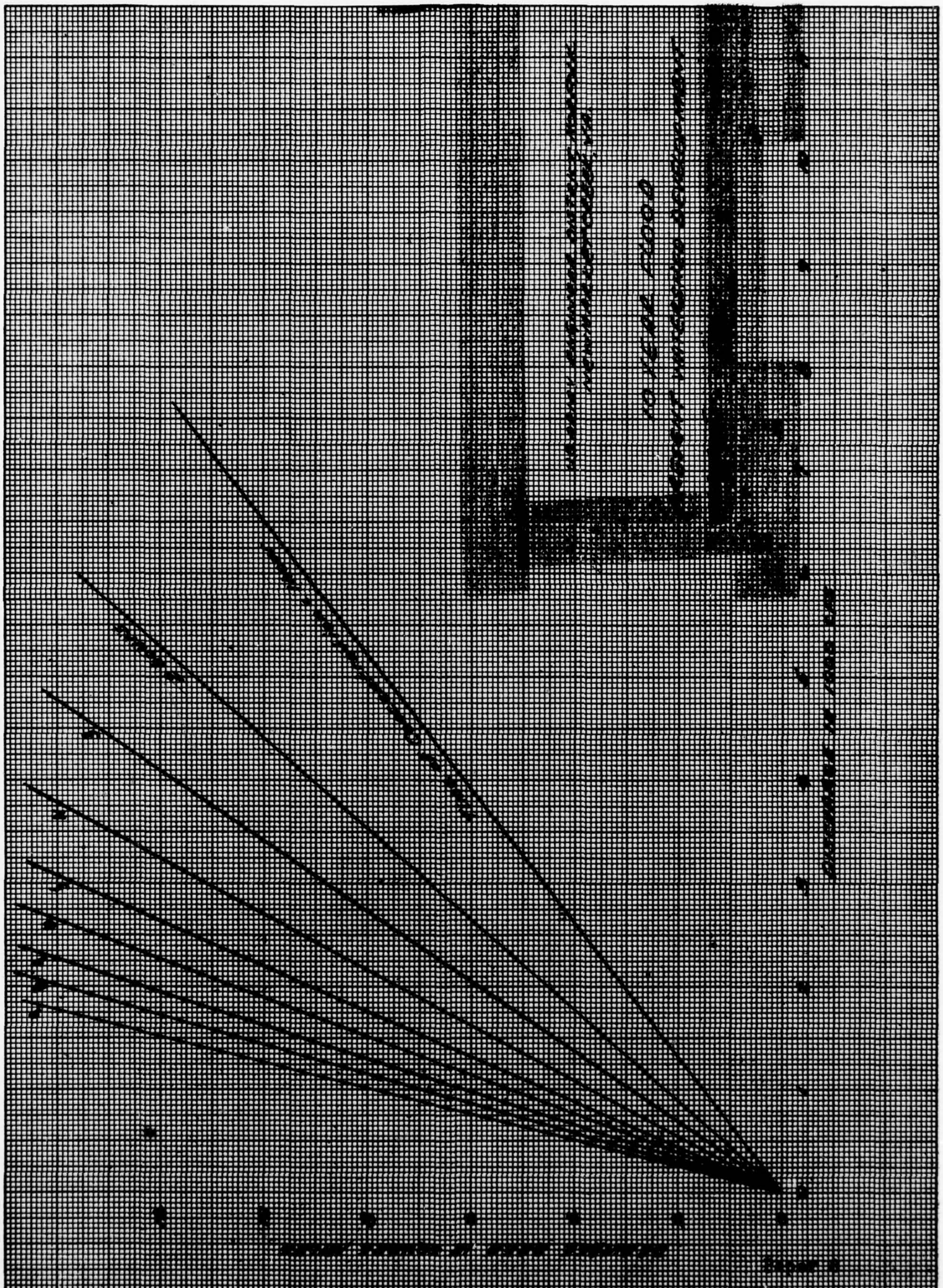


Exhibit 4

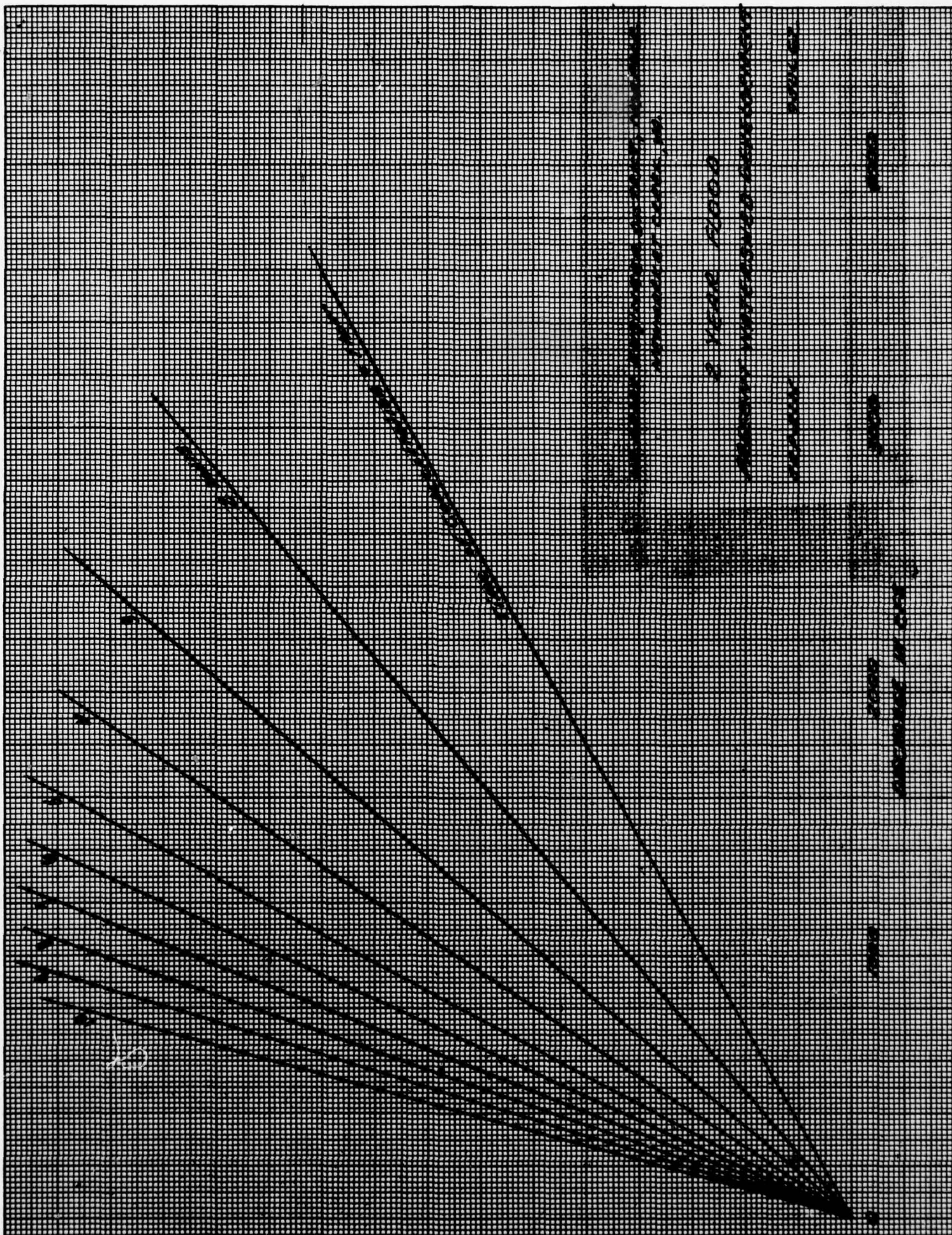


Exhibit 5

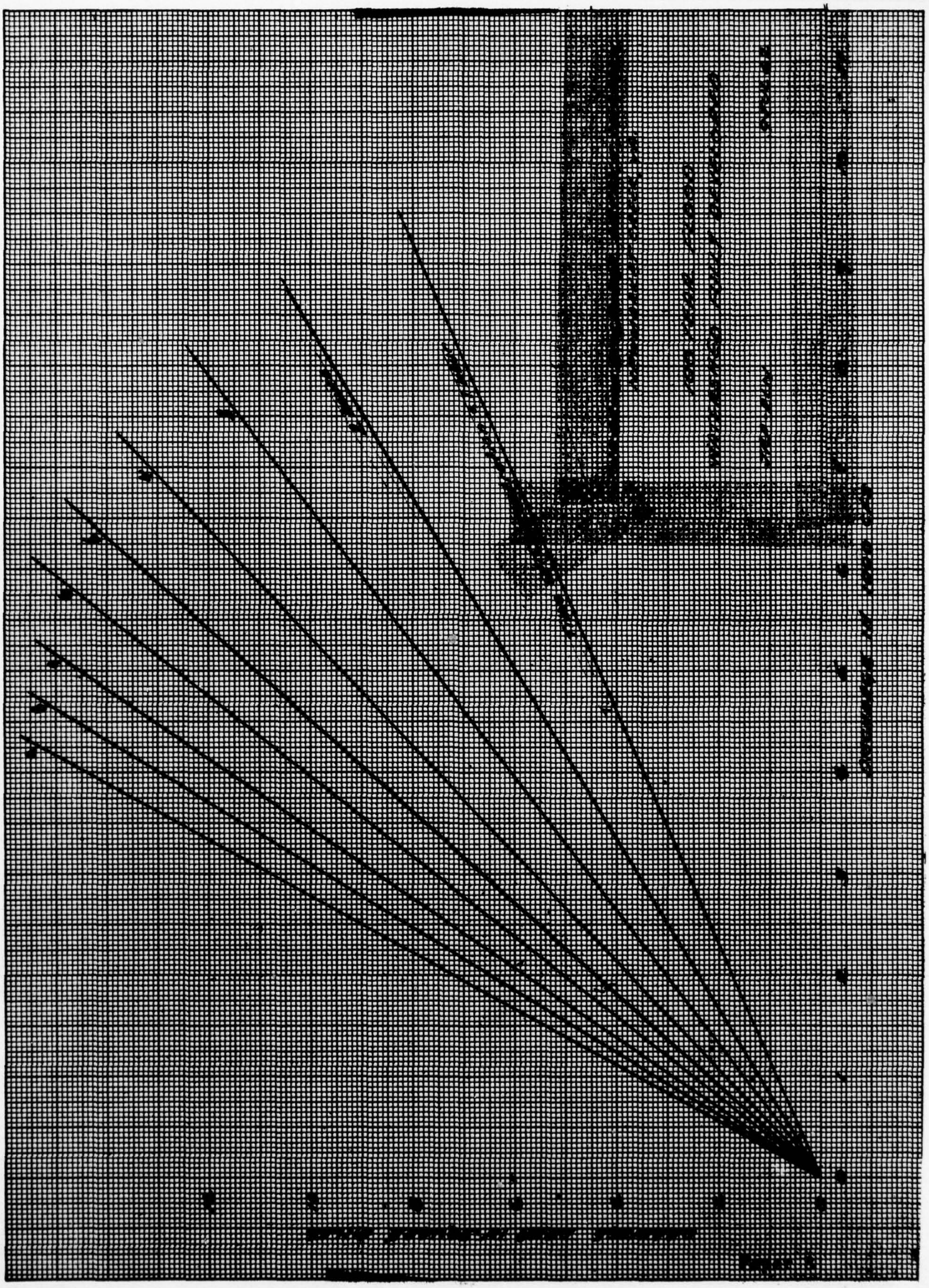


Exhibit 6

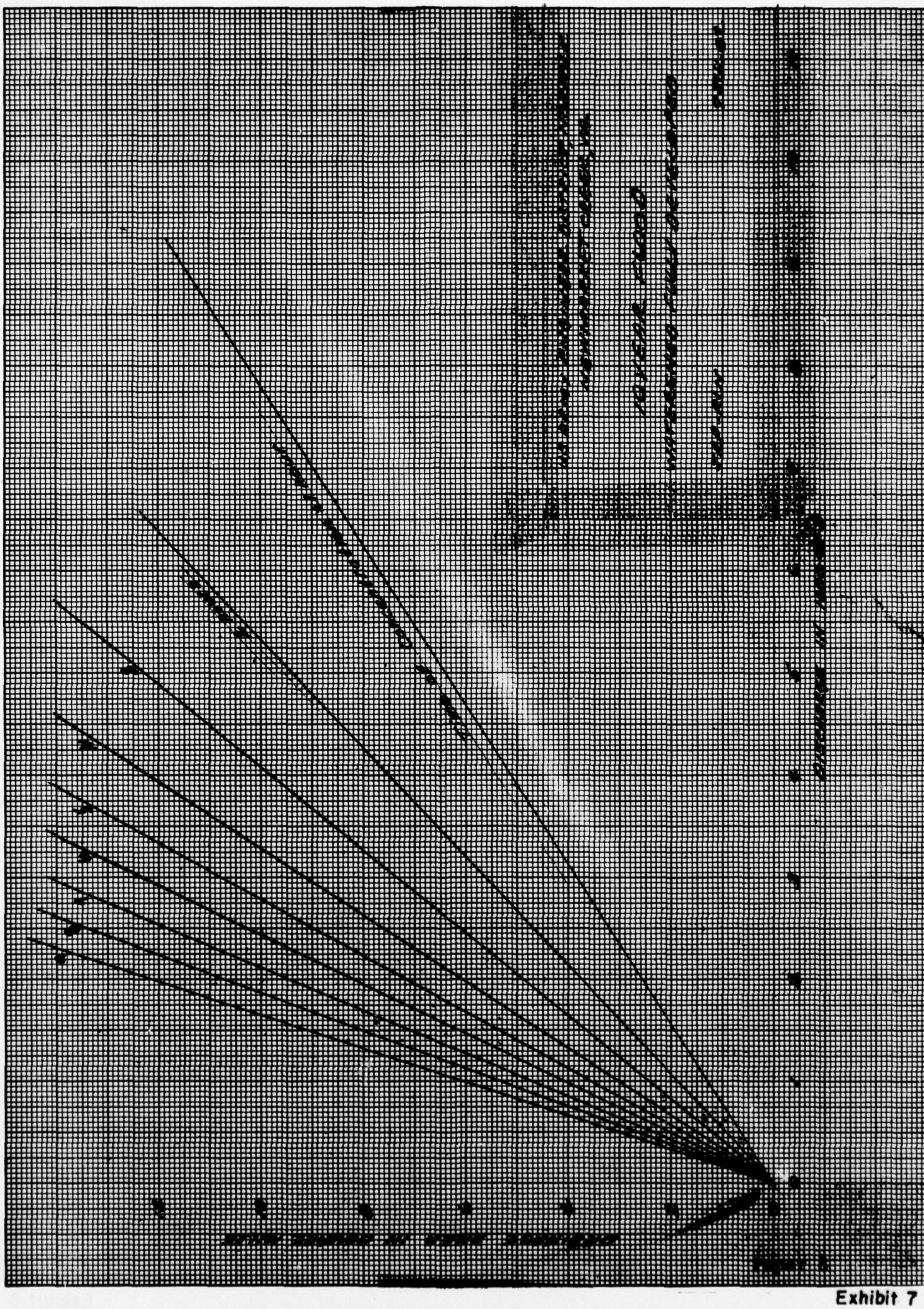


Exhibit 7

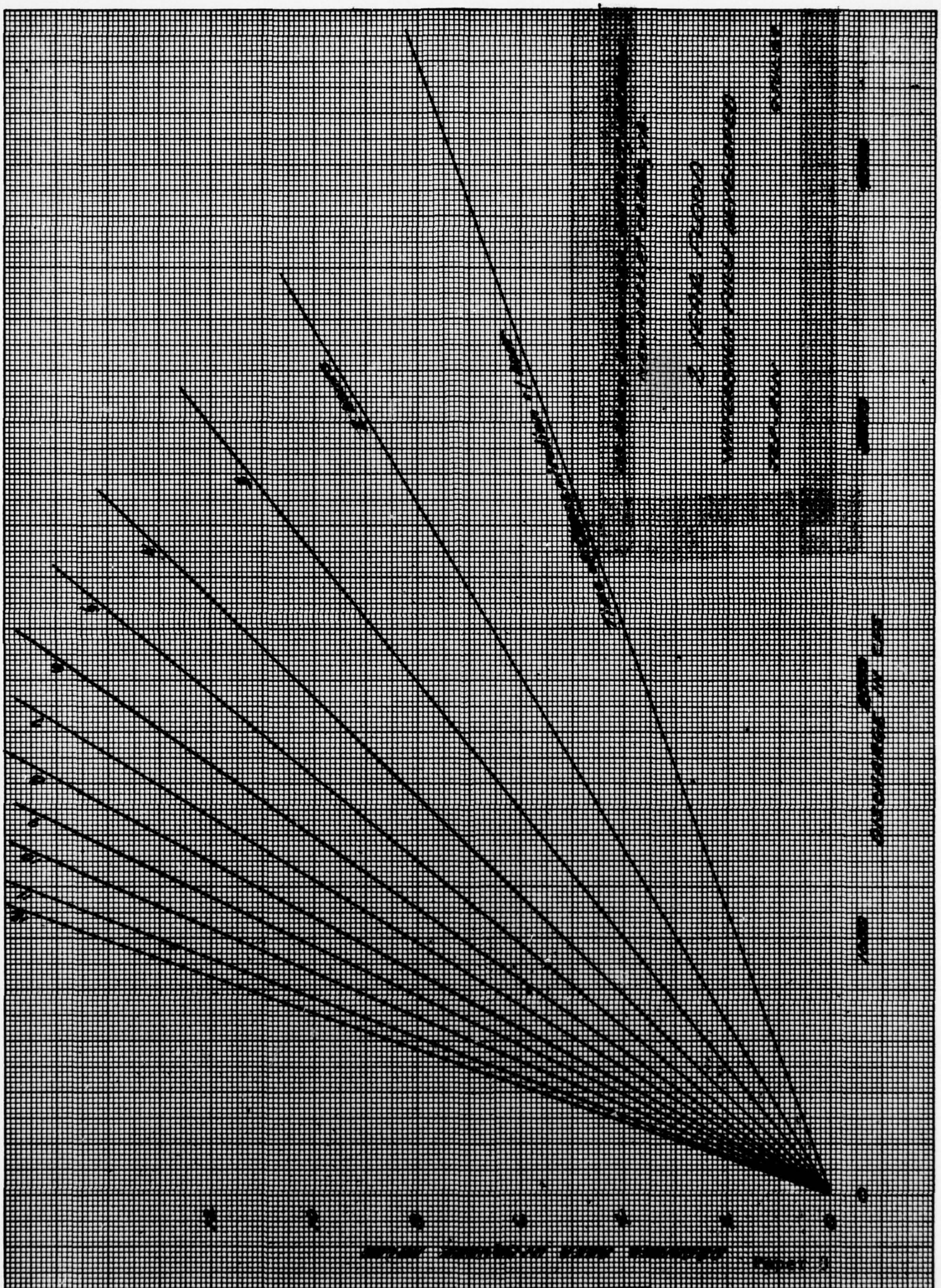


Exhibit 8

URBAN HYDROLOGY IN CONNECTION
WITH CHANNEL IMPROVEMENT PROJECT
AT NEWMARKET CREEK, VIRGINA

Discussion

Question, Mr. Beard: You explained that the longer times of concentration shown in Table 4 result from large overbank flows. Was there some velocity computation made, or are these rough estimates?

Reply, Mr. Matthias: These are rough estimates.

Question, Mr. Hare: Times of concentrations as shown in Table 4 - are these varied with increasing frequency by estimate? How are they changed or what affects the T_c?

Reply, Mr. Matthias: Times of concentration shown in Table 4 are varied with increasing frequency by estimate. It was considered that in the larger floods relatively more water was stored on the flood plain, thus a relatively lower peak flow would result. The time of concentration would be increased somewhat in the larger floods. To some extent the increase in time of concentration was also a tool to obtain a lower peak flow than would occur without the added storage on the overbank areas in larger floods. The time of concentration would also be affected by channel improvement. Thus improvement of a channel would lessen the time of concentration.

Question, Mr. Childs: Apparently there is considerable storage in the overbank low areas. How is this storage evaluated in its effect on the use of the unit hydrograph, and the discharge in the stream?

Reply, Mr. Matthias: The storage is not reflected in the unit hydrograph since the same unit hydrograph was used to compute all sizes of floods. This storage is reflected in the time of concentration as noted in perceding discussion.

Question, Mr. Jones: How did you obtain the difference in discharges from present conditions to fully developed concitions? Was it by changes in rainfall losses only or was the unit hydrograph also changed?

Reply, Mr. Matthias: Difference in discharge from present to fully developed conditions was based on difference in rainfall losses only. The change which would be made by channel improvement was taken into account in selection of a lower time of concentration in entering the curves of drainage area vs discharge as shown in Exhibits 3 through 8.

Question, Mr. Nelson: Do you have in your District's experience any quantitative data to support your assumption that the unit hydrograph peak should be lowered for application to larger floods?

Reply, Mr. Matthias: None, except for computational analysis in the Virginia Beach, Va. area where the unit hydrograph procedure was used to compute flood discharges of various frequencies in an extremely flat area. In the first determination it was found that the volume of water stored under the 100 year profile, was essentially the same as the volume of the computed flood. This indicated that in flat areas, in large floods the amount of water stored is significant. In this situation a lower unit hydrograph peak is indicated in order to arrive at an effective flow at various points along a waterway which would be consistent with the water stored under the profile.

A MATHEMATICAL DETERMINATION
OF THE
ORDINATES OF THE UNIT HYDROGRAPH

by
James A. Constant[1]

Purpose. The purpose of this paper is to develop a mathematical formula which describes the unit hydrograph and to provide a method for determination of the parameters.

Introduction. Unit hydrographs are used by many engineers in the study of flood runoff. A unit hydrograph is defined as the hydrograph generated by one unit (usually one inch) of rainfall excess uniformly distributed over the watershed contributing to the point of unit hydrograph determination. When there is insufficient data to define a natural unit hydrograph, synthetic methods are used. These methods are usually based on those physical properties of the watershed which can be measured by use of a topographic map, and yield as an end result the peak, the time of peak, and the volume under the hydrograph; but, furnish no guide for shaping the unit hydrograph other than methods outlined in EM 1110-2-1405. Previously, the hydrograph was shaped by judgment, passing through the coordinates of the peak and then adjusted until the volume was correct. This procedure is time-consuming and no two people arrive at exactly the same shape hydrograph.

Analysis. Since the distance traveled by every drop of excess rainfall from the point where it fell to the point of unit hydrograph determination is purely random, it seems logical that the equation for runoff

[1]Chief, Reservoir Regulation Unit, Albuquerque District

would be some form of the probability curve. In order that several unit hydrographs might be compared, the coordinates were reduced to dimensionless form and plotted on log-probability graph paper. The percent of total volume was plotted along the probability scale and time divided by the time to 50 percent of the volume was plotted on the logarithmic scale. Such a plot is shown on Figure 1. It will be noted that, when plotted in this manner, the unit hydrograph plots as a straight line. Furthermore, all unit hydrographs will pass through the same point; e.g., where fifty percent of the volume has passed and where $\frac{t}{T}$ equals one. Only the slopes will be different. The general equation for the differential of such a curve is:

$$\frac{d(\Sigma Q)}{d(\log_{10} \frac{t}{T})} = \frac{NW}{\sigma} \phi t \qquad (1)$$

where: $$\phi t = \frac{1}{\sqrt{2\pi}} e^{\frac{-(\log_{10} \frac{t}{T} - M)^2}{2\sigma^2}}$$

ΣQ = accumulated flow
t = time in hours from beginning of runoff
T = time in hours at which 50% of the flow has passed
NW = c.f.s. periods. It is computed by the following formula:

$$NW = 645.33336 \frac{A}{dt}$$

A = contributing area in square miles
σ = standard deviation of the $\log_{10}$ of the values
M = mean of the $\log_{10}$ of the values

$$\frac{1}{\sqrt{2\pi}} = .39894215$$

Since 50 percent of the volume occurs where t = T, then $M = \log_{10}\frac{t}{T} = \log_{10} 1 = 0$ so the term M drops out. Also:

$$d(\log_{10}\frac{t}{T}) = \frac{(\log_{10} e)\frac{dt}{T}}{\frac{t}{T}} = \frac{.43429448(dt)}{t}$$

and: $d(\Sigma Q) = Q$

Putting these values into equation (1):

$$\frac{Q}{.43429448\ \frac{dt}{t}} = \left[\frac{645.3333\ \frac{A}{dt}}{\sigma}\right] .39894215\ e^{\frac{-(\log_{10}\frac{t}{T})^2}{2\sigma^2}}$$

reducing and transposing,

$$Q = \frac{111.8094A}{\sigma t}\ e^{\frac{-(\log_{10}\frac{t}{T})^2}{2\sigma^2}} \qquad (2)$$

Knowing the parameters T and σ, Q can be determined for any given t. Examination of unit hydrographs computed by equation (2) discloses that it satisfies the conditions usually assumed to govern the shape of the unit hydrograph; i.e., it approximately conforms to the W-50 and W-75 curves shown on plate 7 of EM-1110-2-1405, the rising limb is approximately parabolic, and the recession is approximately exponential.

Determination of Parameters. Equation (2) is differentiated and equated to zero to determine the maximum ordinate of the curve.

$$0 = \frac{dQ}{dt} = \left[\frac{-111.8094A}{\sigma t} e^{\frac{-(\log_{10} \frac{t}{T})^2}{2\sigma^2}} \right] \left[\frac{\log_{10} \frac{t}{T}}{\sigma^2 \ln 10} \right] + \left[\frac{-111.8094A}{\sigma t^2} e^{\frac{-(\log_{10} \frac{t}{T})^2}{2\sigma^2}} \right]$$

dividing by the last term,

$$0 = \frac{\log_{10} \frac{t}{T}}{\sigma^2 \ln 10} + 1$$

then:

$$\log_{10} \frac{t}{T} = -2.302585\sigma^2 \tag{3}$$

When Q is at its maximum.

Substituting this value in equation (2),

$$Q\max = \frac{111.8094A}{\sigma\ t_{\max Q}} e^{\frac{-(-2.302585\sigma^2)^2}{2\sigma^2}} = \frac{111.8094A}{\sigma\ t_{\max Q}} e^{-2.650949\sigma^2}$$

rearranging,

$$\frac{e^{-2.650949\sigma^2}}{\sigma} = \frac{t_{\max Q}\ Q_{\max}}{111.8094A} \tag{4}$$

The values on the right side of this equation are the ones usually determined by the various synthetic methods. Note, however, that $t_{\max Q}$ is time from beginning of rainfall excess. If the method being used defines lag as the time from the center of mass of rainfall excess to time of peak, then $t_{\max Q}$ would equal lag plus one-half the unit rainfall excess period.

Equation (4) is not easily solved for σ. A graphical solution is shown in Figure 2. A satisfactory approximation for σ within the range usually encountered, e.g. $.1 < \sigma < .5$, can be obtained by the following equation:

$$\text{Let } y = \log_{10} \frac{t_{maxQ} Q_{max}}{111.8094A}$$

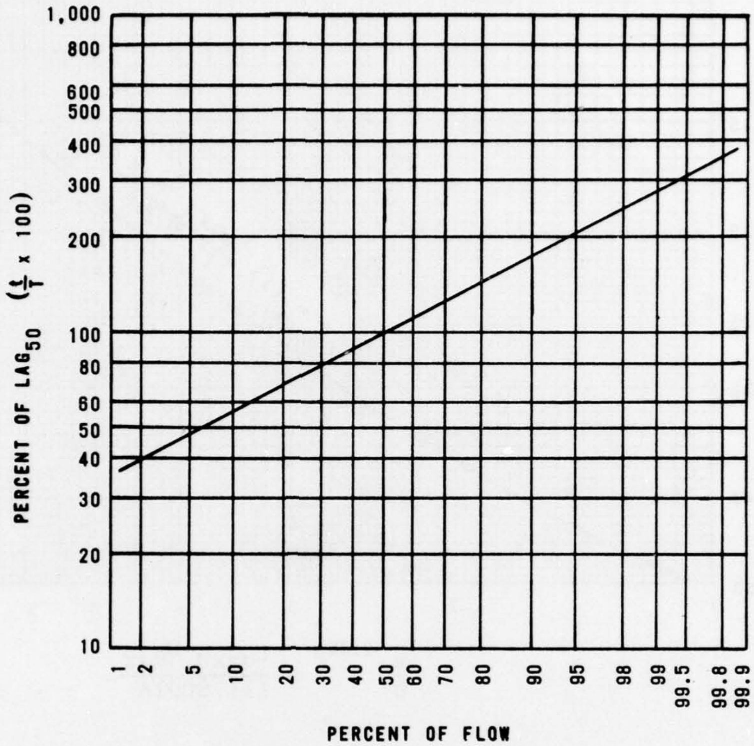
Then:

$$\log_{10} \sigma = -.295133 - .425224y - .242919y^2 - .103322y^3 + .0091443y^4 + .0353098y^5 + .0107619y^6$$

Having determined σ, T is determined by rearranging equation (3) in the form:

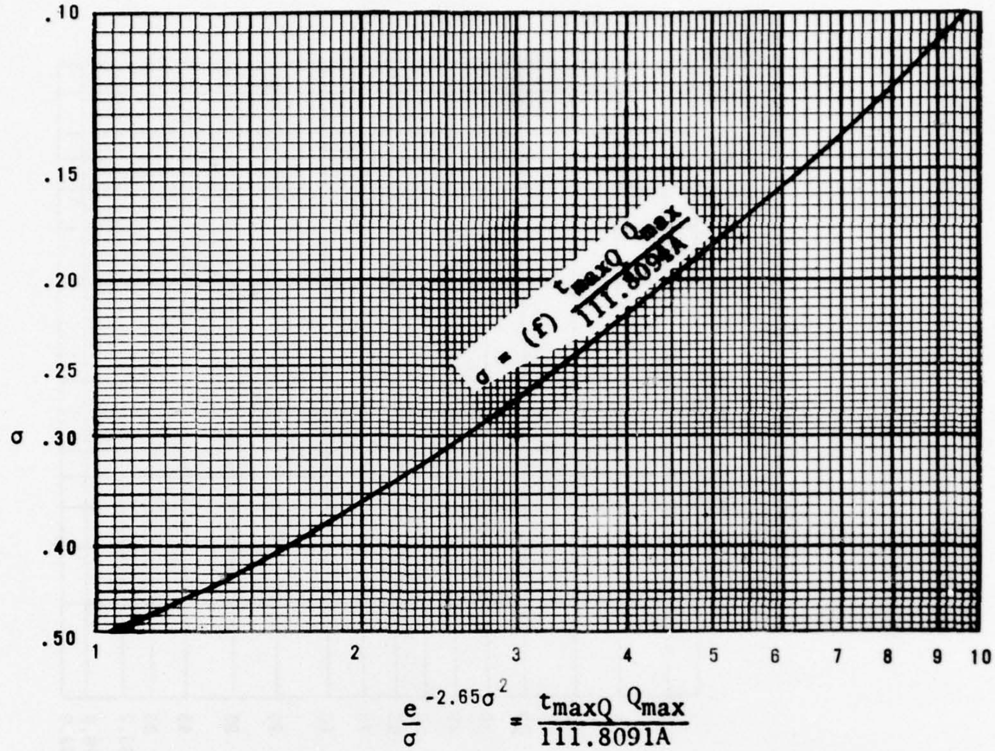
$$\log_{10} T = 2.302585\sigma^2 + \log_{10} t_{maxQ} \tag{5}$$

Conclusion. An equation for the unit hydrograph has been developed and a means of evaluating the parameters in terms of the time and magnitude of the unit hydrograph peak and the contributing area have been presented. The method is easily programmed for use in an electronic computer as a program in itself or as part of a larger program, such as one for determining basin runoff during flood periods. A typical unit hydrograph computed by this means is shown in Figure 3; this is the same unit hydrograph shown in Figure 1.



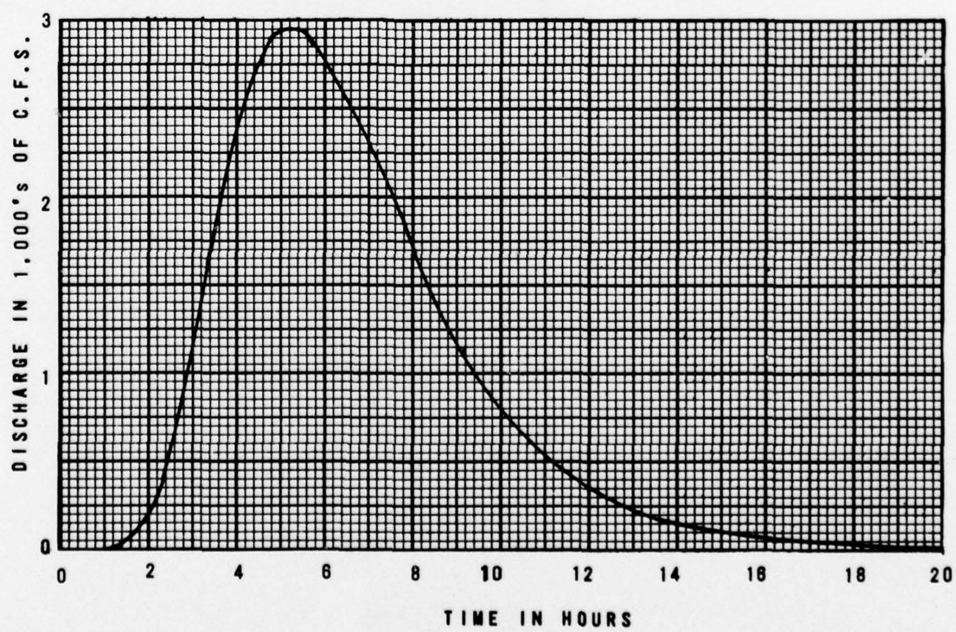
1,000
800
600
500
400
300
200
100
80
60
50
40
30
20
10
PERCENT OF LAG$_{50}$ $(\frac{t}{T} \times 100)$
1
2
5
10
20
30
40
50
60
70
80
90
95
98
99
99.5
99.8
99.9
PERCENT OF FLOW

FIG. 1



.10
.15
.20
.25
σ .30
.40
.50
1
2
3
4
5
6
7
8
9
10
σ = (f) $\frac{t_{maxQ}\ Q_{max}}{111.8091A}$
$\frac{e^{-2.65\sigma^2}}{\sigma} = \frac{t_{maxQ}\ Q_{max}}{111.8091A}$

FIG. 2



DISCHARGE IN 1,000's OF C.F.S.
0
1
2
3
0
2
4
6
8
10
12
14
16
18
20
TIME IN HOURS

FIG. 3

APPENDIX

EXAMPLE
FOR
COMPUTING THE ORDINATES OF THE UNIT HYDROGRAPH

UNIT HYDROGRAPH FOR SUBAREA NO. 1 OF SALUDA RIVER, S.C.
(SEE PLATE 10 OF EM 1110-2-1405)

DA = 970 sq. mi. t_R = 6 hrs. t_{pR} = 28 hrs. W_{75} = 27 hrs.

Q_p = 12,800 c.f.s. q_{pR} = 13.2 c.f.s./sq.mi. W_{50} = 44 hrs.

Based on these data:

$$Q_{max} = 12{,}800 \text{ c.f.s.} \quad t_{maxQ} = t_{pR} + \frac{t_R}{2} = 28 + \frac{6}{2} = 31.0 \text{ hrs.}$$

Using Equation (4)

$$\frac{e^{-2.65\sigma^2}}{\sigma} = \frac{t_{maxQ}\,Q_{max}}{111.8094A} = \frac{396{,}800}{108{,}455} = 3.6586$$

$\sigma = 0.235$ (first approximation from figure 2)

$= 0.23585$ (by successive approximation)

Using Equation (5)

$$\log_{10} T = 2.302585\sigma^2 + \log_{10} t_{maxQ}$$

$$= 2.302585(.055625) + \log_{10} 31.0$$

$$= 0.12808 + 1.49136$$

$$= 1.61944$$

Then:

$T = 41.633$ hours

Equation (2) becomes:

$$Q = \frac{111.8094(970)}{0.23585\,t} e^{\frac{-\left(\log_{10}\frac{t}{41.63}\right)^2}{2(0.23585)^2}} = \frac{459{,}846.26}{t} e^{\frac{-(\log_{10} t - 1.61944)^2}{0.11125}}$$

TABLE I

Col. No. 1 t (hrs.)	2 $\frac{459,846.26}{t}$	3 $\log_{10} t$	4 Col. 3 -1.61944	5 $\frac{-(\log_{10} \frac{t}{T})^2}{0.11125}$	6 $e^{\frac{-(\log_{10} \frac{t}{T})^2}{0.11125}}$	7 Col. 2 x Col. 6 c.f.s.
0						0
3	153,282	0.47712	-1.14232	-11.72939	0.00001	1
6	76,641	0.77815	-0.84129	- 6.36200	0.00173	133
9	51,094	0.95424	-0.66520	- 3.97745	0.01874	956
12	38,321	1.07918	-0.54026	- 2.62365	0.07253	2,779
15	30,656	1.17609	-0.44335	- 1.76682	0.17084	5,237
18	25,547	1.25527	-0.36417	- 1.19209	0.30358	7,756
21	21,897	1.32222	-0.29722	- 0.79406	0.45203	9,898
24	19,160	1.38021	-0.23923	- 0.51444	0.59784	11,455
27	17,031	1.43136	-0.18808	- 0.31797	0.72760	12,392
30	15,328	1.47712	-0.14232	- 0.18207	0.83354	12,776
31	14,834	1.49136	-0.12808	- 0.14746	0.86290	12,800
36	12,774	1.55630	-0.06314	- 0.03584	0.96480	12,324
42	10,949	1.62325	+0.00381	- 0.00013	0.99987	10,948
48	9,580	1.68124	0.06180	- 0.03433	0.96625	9,257
54	8,516	1.73239	0.11295	- 0.11468	0.89164	7,593
60	7,664	1.77815	0.15871	- 0.22642	0.79738	6,111
66	6,967	1.81954	0.20010	- 0.35991	0.69773	4,861
72	6,387	1.85733	0.23789	- 0.50869	0.60128	3,840
78	5,895	1.89209	0.27265	- 0.66821	0.51262	3,022
84	5,474	1.92428	0.30484	- 0.83530	0.43374	2,374
90	5,109	1.95424	0.33480	- 1.00756	0.36511	1,865
96	4,790	1.98227	0.36283	- 1.18333	0.30595	1,465
102	4,508	2.00860	0.38916	- 1.36131	0.25632	1,155
108	4,258	2.03342	0.41398	- 1.54049	0.21437	913
114	4,033	2.05690	0.43746	- 1.72019	0.17888	721
120	3,832	2.07918	0.45974	- 1.89987	0.14958	573
126	3,650	2.10037	0.48093	- 2.07904	0.12505	456

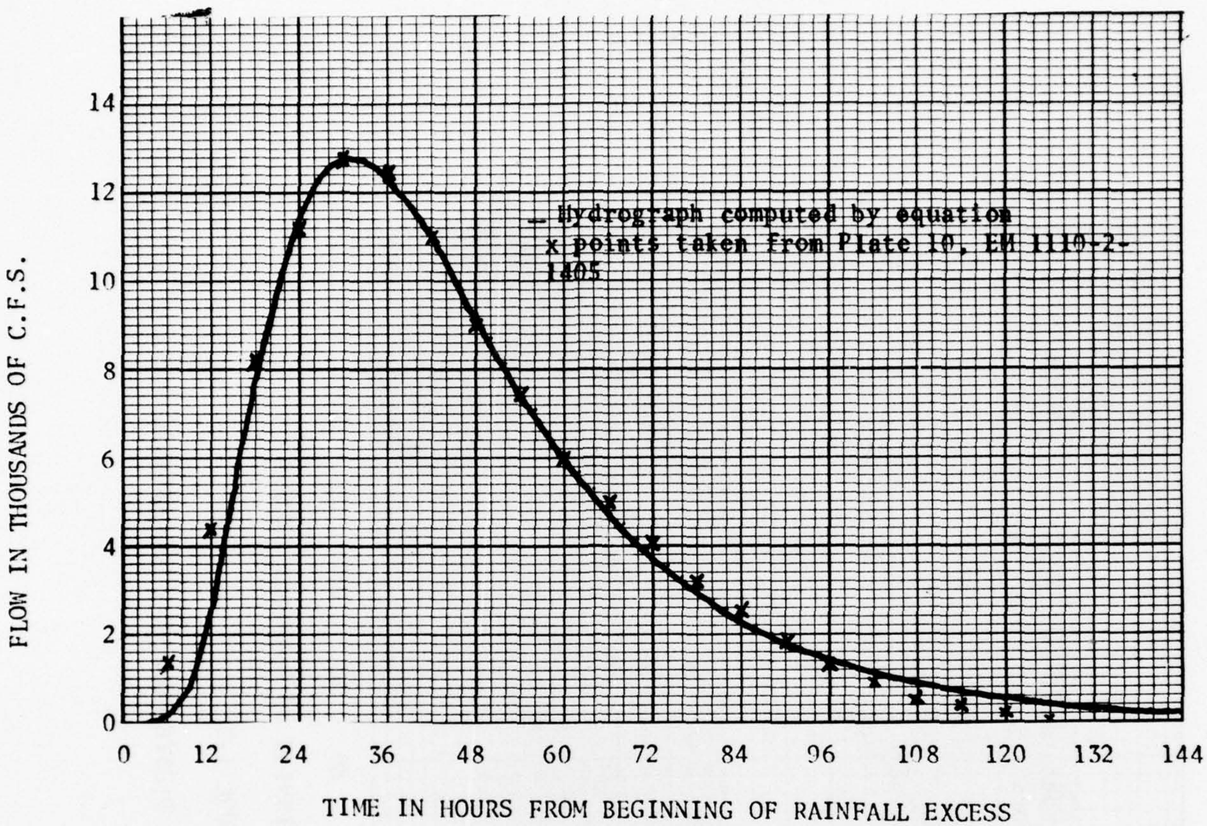


FIGURE 1-1
COMPUTED UNIT HYDROGRAPH

UNIT HYDROGRAPH CHARACTERISTICS

	Data From EM 1110-2-1405	Data For Computed Hydrograph
Area (sq.mi.)	970	970
T_R (hrs.)	6	6
t_{pR} (hrs.)	28	28
t_{maxQ} (hrs.)	31	31
640 C_p	370	370
Q_p (c.f.s.)	12,800	12,800
q_p (c.f.s.)	13.2	13.2
W_{75}	27	26.3
W_{50}	44	42.4
σ	--	.23585
T (hrs.)	36.745	41.634

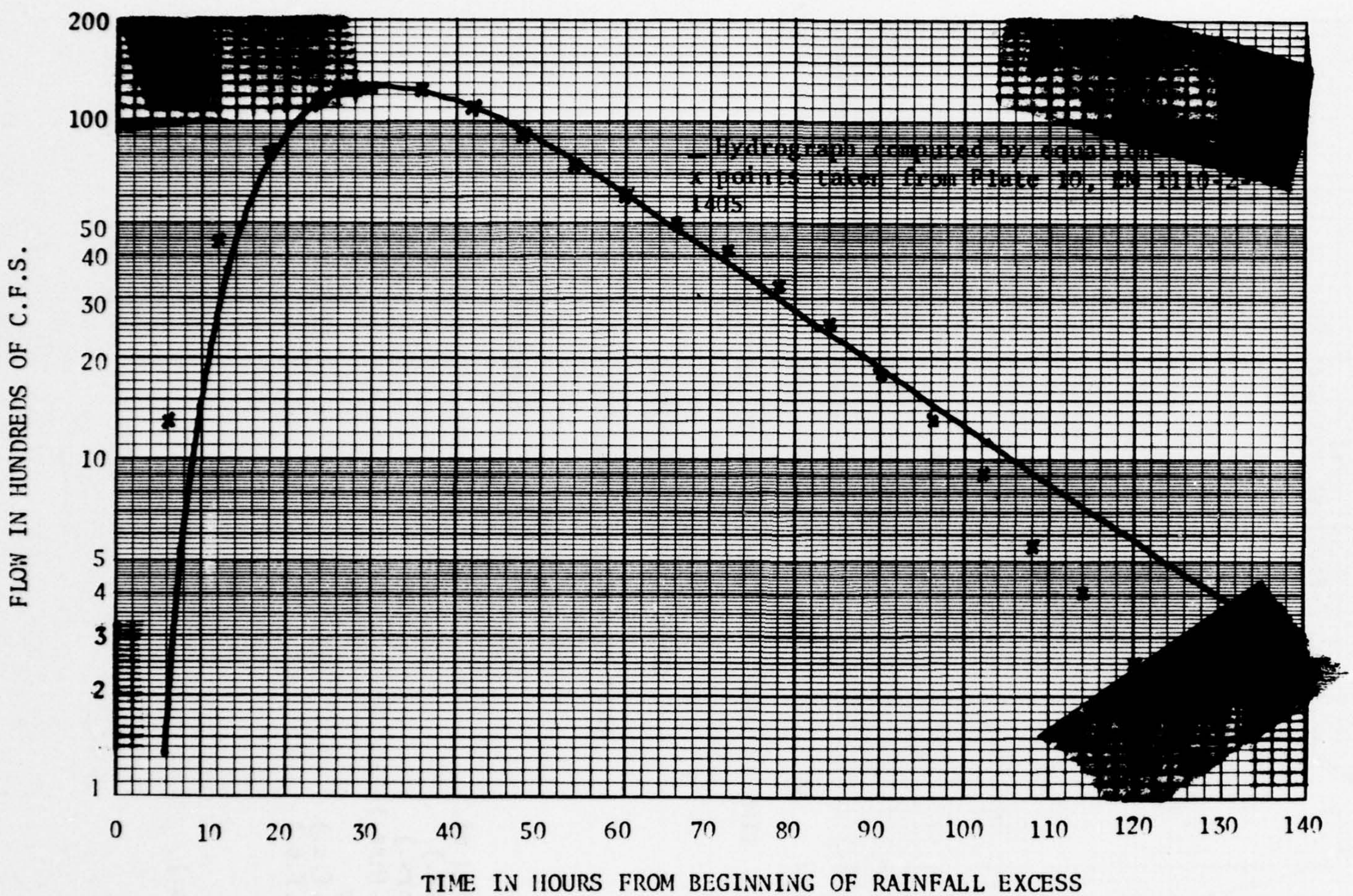


FIGURE 1-2

HYDROGRAPHS SHOWN IN FIG. 1-1 PLOTTED ON SEMI-LOGARITHMIC SCALE

A MATHEMATICAL DETERMINATION OF THE
ORDINATES OF THE UNIT HYDROGRAPH

Discussion

Question, Mr. Beard: One advantage of your technique is that it enables direct computation of unit hydrographs in the computer. Are there other advantages? Do you consider that a method such as the Clark method that uses the basin time-area curve would have any substantial advantage over the method that you describe?

Reply, Mr. Constant: One advantage is that it yields consistent results when used by the novice as well as the experienced engineer.

The establishment of a basin time-area curve can only be based on "engineering judgment", - a quality which varies with time and with the individual. If hydrology is to move out of the realm of art and become a science, problem solving must depend on more than judgment. Areas which are suspected to produce a unit hydrograph which does not conform to the normal shape as defined by EM 1110-2-1405 should be subdivided into two or more areas, each of which has a drainage pattern which can be expected to conform to the normal shape.

Question, Mr. Holler: From your statement that, "The distance travelled by every drop of excess from the point where it fell to the point of unit hydrograph determination is purely random", it follows that some type of distribution is involved. Which distribution? Normal? Exponential? Poisson? etc.

Reply, Mr. Constant: No attempt was made to establish the type of distribution of the original variates. Instead, the variates were transformed to fit a log-normal distribution, since this is the one most familiar to hydrologic engineers.

AN ANALYSIS OF THE EFFECTS OF URBANIZATION ON UNIT HYDROGRAPH CHARACTERISTICS ANTELOPE CREEK BASIN - LINCOLN, NEBRASKA

by Keith A. Johnson[1]

1. Introduction

The Antelope Creek studies described in this paper were made in 1960 for the purpose of obtaining hydrologic design requirements and hydrologic effects of a small dam being studied in the Antelope Creek basin. The proposed dam was located in the upper portion of the basin to control runoff from a 5.4 square mile rural type area. City Planners had forecast, however, that in the next 50 years the area above the dam would become a fully developed urban area. The results of this study were used in evaluating future flood probability conditions in the basin under the anticipated urban development with and without the dam in place. The hydrologic design requirements of the dam were also based on the projections of urbanization in the upstream area.

This paper will be confined to a discussion of the unit hydrograph analysis made for the Antelope Creek study. The determination of unit graph characteristics for the urban and rural portions of the basin is presented. In addition, the results obtained in the Antelope Creek study are compared with the results of studies that have become available subsequent to 1960.

2. Basin Description

Antelope Creek is a 13 square mile watershed located in the Salt Creek basin in Eastern Nebraska. It empties into Salt Creek within the city limits of Lincoln, Nebraska. The upper 7.05 square miles of the basin drains a rural area where land use is devoted to agricultural purposes. The lower 5.9 square miles of the basin is located within the Lincoln city limits and is

[1]Chief, Hydrology and Meteorology Section, Omaha District

essentially a fully developed urban area. Topography of the basin varies from moderately to steeply rolling hill land with well defined stream channels located in bowl-shaped valleys. The basin is about 8 miles long and 2.5 miles wide at the broadest point. Slope of the main stream varies from 50 feet per mile in the upper 3.4 miles of channel to 20 feet per mile in the lower 6.8 miles.

Antelope Creek itself flows in a northwesterly direction and has a total stream length of 10.2 miles. Above the city limits the channel is an eroded gully-type drain with variable capacity. As it enters the city limits it flows into a shaped ditch with reasonably uniform bottom widths and side slopes. This channel continues for about 3 miles to where it flows into a closed conduit which continues for 0.7 miles. Below the closed conduit the flow is carried in an open concrete-lined channel which extends for 0.5 miles to where it empties into Salt Creek. A map of the basin is shown on plate 1. Within the city limits channel capacity in the improved channel section above the closed conduit is about 2,000 c.f.s., which is also the capacity of the closed conduit. Below the conduit, in the concrete lined section, channel capacity is considerably in excess of 2,000 c.f.s.

3. Stream Gaging Records

A stream gaging station was established at a point about 0.5 mile above the mouth of Antelope Creek in June 1958. Drainage area of the basin above the gaging station is 12.55 square miles. Data collected at this station between June 1958 and October 1960 were used in the flood hydrograph analyses for Antelope Creek.

4. Runoff Characteristics

Floods on Antelope Creek are caused by runoff from high intensity thunderstorm rainfall. Peaking times in the basin are short and records indicate that base length of surface runoff hydrographs is about 12 hours for runoff events from storms covering the entire basin. The records also indicate a significant double peak characteristic in flood runoff hydrographs.

5. Flood Hydrograph Analysis

Four flood hydrographs were analyzed in order to develop unit hydrograph characteristics for the basin. These hydrographs, together with other pertinent data, are shown on plates 2 to 5. Distribution of the rainfall data shown on the plates was based on the recorder record from the Lincoln station. The double peak characteristic of the basin is demonstrated quite clearly from the floods of 3-4 and 5-6 September 1958 shown on plates 3 and 4.

The flood hydrograph analysis was made assuming that the first peak shown on the hydrographs was caused by runoff from the 5.5 square mile urbanized portion of the basin. The second peak was assumed to originate from the 7.05 square miles of rural area located in the upper portion of the watershed. Using this assumption the four hydrographs shown on plates 2 to 5 were divided to reflect runoff contributions from the urban and rural portions of the basin. This division was made on a trial and error basis, although uniformity was strived for in such things as base length and peaking times. The hydrograph contributions determined for each portion of the basin are also shown on plates 2 to 5.

6. Unit Hydrographs

From the flood hydrographs discussed in the previous paragraph, unit hydrographs were determined for the urban and rural portions of the basin.

The results of these computations are shown on plate 6. As indicated on this plate the beginning of runoff for unit hydrographs from the rural portion of the basin reflects travel time through the 4 miles of channel lying between the downstream point of the rural area and the gaging station.

7. Reconstitution

The average unit hydrographs determined for each portion of the basin shown on plate 6 were tested by reconstituting the flood of 9-10 July 1958. This reconstitution is shown on plate 7. As indicated from data shown on this plate, the heaviest rainfall was centered in the rural portion of the area. Both the observed and reconstituted hydrographs reflect the observed rainfall distribution.

8. Unit Hydrograph Characteristics

Urbanized Area. The average 1-hour unit hydrograph for the 5.5 square mile urbanized portion of Antelope Creek is shown on plate 6. As indicated on the plate, the Snyder's constants from this unit graph are $C_t = 0.60$ and $C_p = 0.80$ ($640\ C_p = 512$). The unit graph peak is 2,800 c.f.s. and the peaking time is 1.25 hours.

Rural Area. The 1-hour constants computed from the average unit hydrographs shown on plate 6 for the 7.05 square mile rural area are C_t · 1.14 and $C_p = 0.73$ ($640\ C_p = 467$). It was assumed in this computation that the average unit hydrograph for the rural area was at a point 4 miles upstream from the gaging station, which is the lower end of the rural area.

9. Unit Graph Comparisons - Urban Area to Rural Area

The indicated effects of urbanization on unit hydrographs in the Antelope Creek basin were obtained by using the constants developed from the urbanized portion of the basin ($C_t = 0.50$, $C_p = 0.80$) to compute a unit hydrograph for the 7.05 square mile rural area of the basin. This unit graph is shown on plate 8 in comparison with the unit hydrograph developed for the area in its present rural state. As shown on the plate, the peaking time of the 1-hour unit hydrograph computed from the urbanized constants is about half as long as the rural area unit hydrograph and the peak discharge was increased by 149 percent. Similar differences are indicated for the 30-minute unit hydrographs shown on plate 8.

10. Comparison with Other Studies

At the time the Antelope Creek study was made there was no opportunity to compare the results with results of other studies on urbanized areas. However, subsequent to this study, which was made in 1960, the results of other studies concerning the effects of urbanization on unit hydrograph characteristics have been published. Two of these studies have been selected for comparison with the results of the Antelope Creek study. One is the technical report of the University of Texas to the Texas Water Commission dated July 1965, entitled, "A Study of Some Effects of Urbanization on Storm Runoff from a Small Watershed." The other study was a Criteria Manual on Urban Storm Drainage prepared for the Denver Regional Council of Governments by the consulting engineering firm of Wright-McLaughlin of Denver, Colorado.

Texas Water Commission Report

Generalized equations were published in this report for developing 30-minute unit hydrographs for rural and urban areas. These equations for peaking time and for peak discharge are given below.

FOR RURAL WATERSHEDS

$$T_{RR} = 2.65\ L^{0.12}\ S^{-0.52}$$

$$Q_R = 1.70 \times 10^3\ A^{0.88}\ T_{RR}^{-0.30}$$

FOR URBAN WATERSHEDS

$$T_{RU} = 20.8\ \Phi\ L^{0.29}\ S^{-0.11}\ I^{-0.61}$$

$$Q_U = 1.93 \times 10^4\ A^{0.91}\ T_{RU}^{-0.94}$$

where T_{RR} = Time from beginning of runoff to peak of unit hydrograph for rural watershed (in minutes).

L = Length of stream (in feet).

S = Difference in elevation from headwaters of basin to downstream point divided by length (in feet per foot).

Q_R = Peak discharge for rural watershed in c.f.s.

A = Drainage area in square miles.

T_{RU} = Time from beginning of runoff to peak of unit hydrograph for urban watershed (in minutes).

Q_U = Peak discharge for urban watershed in c.f.s.

Φ = A factor to account for reduction in peaking time due to channel improvements or addition of storm sewers (varies from 0.6 to 1.0).

I = The percent of impervious area.

Unit hydrograph peaking times and peak discharge values were computed for the 7.05 square mile rural portion of the Antelope Creek basin using these equations.

The computation was made assuming the present rural development and, for comparison, assuming various degrees of urbanization in the area. The results are summarized below.

ASSUMING RURAL DEVELOPMENT

Drainage Area (sq.mi.)	L (feet)	S (ft/ft)	T_{RR} (Min)	Q_R (c.f.s.)
7.05	29,000	.0076	115	2,270

ASSUMING URBAN DEVELOPMENT

Drainage Area (sq.mi.)	L (feet)	S (ft/ft)	Φ	I (percent)	T_{RU} (Min)	Q_U (c.f.s.)
7.05	29,000	.0076	0.80	40	59	2,450
			0.60	50	39	3,680
			0.60	60	34	4,090

These results indicate that depending on the degree of urban development, the peaking time of the 30-minute unit hydrograph for the Antelope Creek area, if urbanized, would be reduced by 49 to 70 percent from the rural area peaking time and the peak discharge would be increased by anywhere from 8 to 80 percent. As shown on plate 8 the 30 minute unit hydrographs from the Antelope Creek study indicated a 50 percent reduction in peaking time from rural to urban, but an increase in peak discharge of 153 percent.

Wright-McLaughlin Study

In Volume 1 of the Wright-McLaughlin Criteria Manual on Urban Storm Drainage the following Snyder's constants are suggested for urbanized areas in the Denver region.

Table 1
Suggested C_p and C_t Values for the Denver Region

Percent Impervious	Values for Average Conditions	
	C_t (Note 1)	C_p (Note 2)
60	.25	.45
40	.30	.50
20	.35	.55

Note 1. Add 10% for sparsely sewered areas. Subtract 10% for fully sewered areas.

Add 10% for very flat basins. Subtract 10% for steep basins.

Note 2. Subtract 10% for sparsely sewered areas; add 10% for fully sewered areas.

Subtract 10% for very flat basins; add 10% for steep basins.

These constants are indicated as being for a unit runoff duration of 5 to 10 minutes. They were applied to the Antelope Creek rural area and the resulting unit hydrographs were converted to 30-minute unit duration. The conditions studied ranged from 20 percent impervious and sparsely sewered to 60 percent impervious and fully sewered. The results are summarized below.

Table 2
Antelope Creek
30-Minute Unit Hydrograph Values for 7.05 Square Mile Rural Area

Condition	Peaking Time (min)	Peak Discharge (c.f.s.)
20% Impervious Sparsely Sewered	65	2,580
40% Impervious Average Conditions	55	3,090
60% Impervious Fully Sewered	50	4,000

The effects indicated by this study in going from a low to a high degree of urban development are a 23 percent reduction in peaking time for the 30-minute unit hydrograph and a 55 percent increase in peak discharge.

SUMMARY

For comparison purposes the results obtained from the three methods presented are summarized below:

Table 3
Comparison of 30-Minute Unit Hydrograph Values

Study	Peaking Time (min)		Percent Reduction	Peak Discharge (c.f.s.)		Percent Increase
	Rural	(1) With Urbanization		Rural	(1) With Urbanization	
Antelope Creek	160	80	50	1,520	3,850	153
Texas Water Comm	115	34	70	2,270	4,090	80
Wright-McLaughlin(2)	65	50	23	2,580	4,000	55

(1) The unit graph values computed for the urbanized portion of Antelope Creek are assumed to reflect a high degree of urban development. The values selected from the other studies for comparison therefore represent the maximum urban development considered.

(2) The values shown as rural from the Wright-McLaughlin study are for the 20 percent impervious and sparsely sewered condition.

DISCUSSION

It is not intended to present any real definitive conclusions from the data presented in this paper. The increase in unit hydrograph peaks as area is converted from rural to urban usage seems to be a generally accepted assumption. However, the magnitude of the increase would appear to be a logical subject for research. The study made by the University of Texas

was unquestionably a significant contribution. There are probably others which I have not had the opportunity to review. The effects of urbanization obtained from using the generalized equations in the Texas report are, however, significantly less than you might expect from reading the conclusions in this report. For the Antelope Creek Area, the use of the generalized equations resulted in an increase in unit hydrograph peak discharge of 80 percent in going from a rural area to one of 60 percent impervious and fully sewered. In the study watershed used in the Texas report (Waller Creek watershed) the equations produced increases in unit graph peaks of 62 to 66 percent in comparing the rural area with a 50 percent impervious urban area and partially sewered. These increases appear to contradict the conclusions of the report, which state that urbanization in the Waller Creek watershed has already increased unit hydrograph peaks from 200 to 260 percent, and that future development to 50 percent impervious cover will increase unit peaks 330 percent over rural conditions.

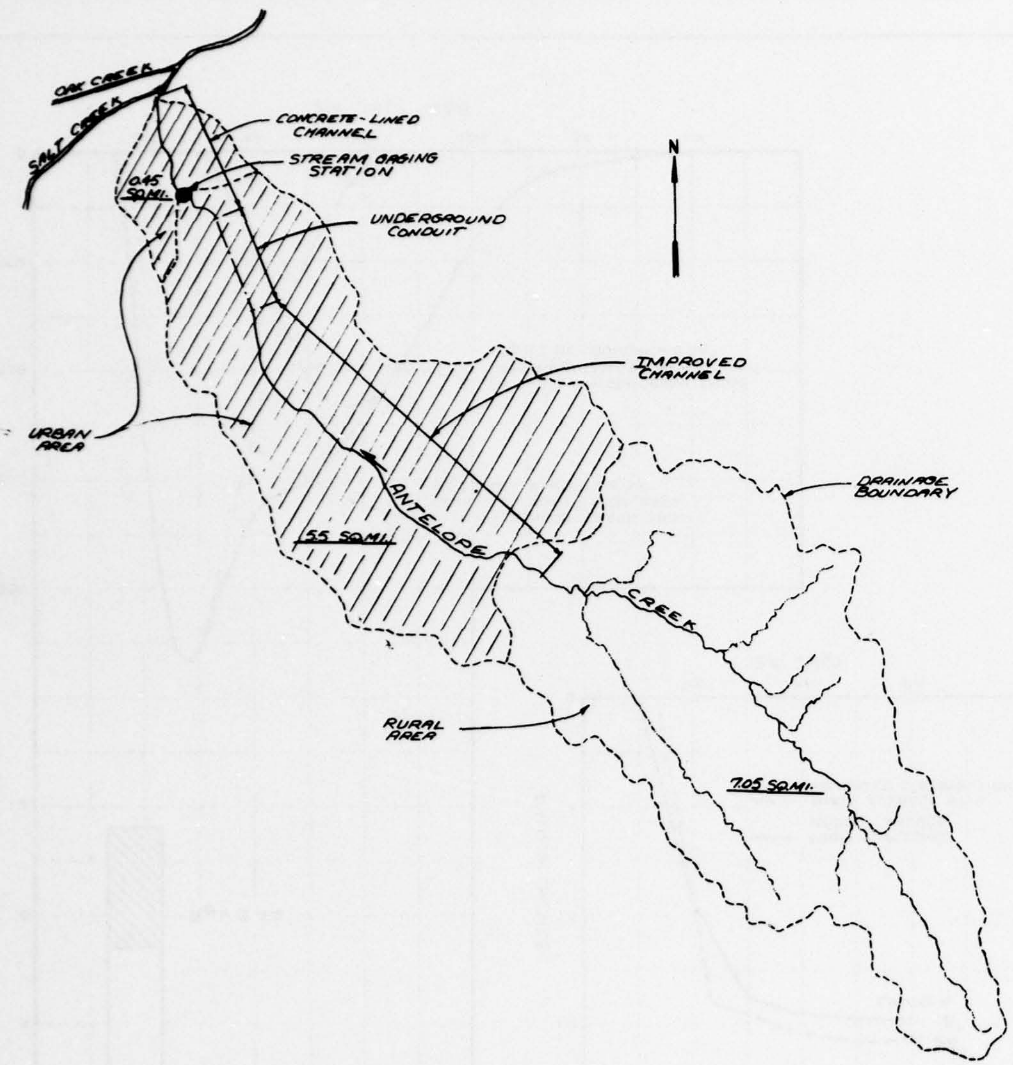
Another comment that seems pertinent concerns the effect on unit graph values that result from the introduction of sewering and channel improvement factors. In the Texas study the factor Φ is indicated as varying from 1.0 for natural conditions with no development to 0.60 for areas with fully developed storm sewer systems and extensive channel improvements. In the Antelope Creek area the effect of the Φ factor is shown by comparing results of applying the generalized equations with varying factors of Φ.

Table 4
Unit Hydrograph Results With Varying Values of Φ

Condition	Unit Hydrograph Peaking Time (min)	Unit Hydrograph Peak Discharge (c.f.s.)
Rural	115	2,270
Urban - 60% impervious $\Phi = 1.0$	57	2,530
Urban - 60% impervious $\Phi = 0.80$	46	3,100
Urban - 60% impervious $\Phi = 0.60$	34	4,090

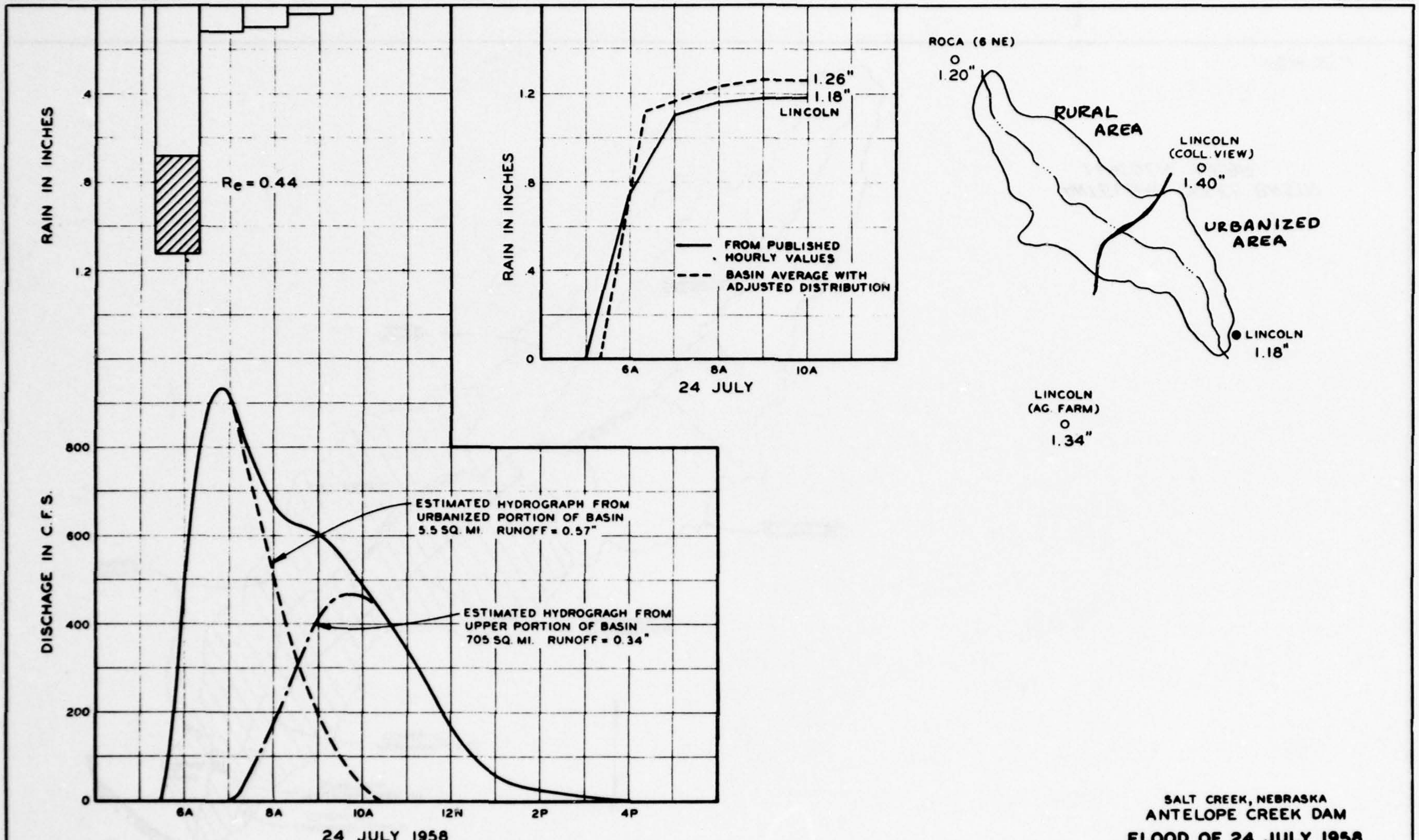
These results indicate that the most important factor by far in determining the effects of urbanization on peak discharges is an estimate of the storm sewer and channel improvement factor. Similar results are indicated in applying the storm sewering adjustments recommended in the Wright-McLaughlin study. It seems likely that the judgment used in selecting the storm sewer and channel improvement factor would vary from person to person with a resulting variability in computation results. Proper recognition of the effects produced in applying the storm sewering factor should therefore be understood by those who are using the equations. This will permit the investigator to better evaluate the results obtained.

There is a general agreement in the urbanized unit graph peak values determined from the methods investigated in this study (see table 3 on page 9). This may be a happy coincidence, but it does provide a degree of comfort in being able to obtain similar results from three different methods. The study presented in the Texas Water Commission report is believed to be one of the more comprehensive studies made concerning the effects of urbanization on unit hydrograph characteristics. The generalized equations presented in this report should be a useful tool in evaluating these effects.

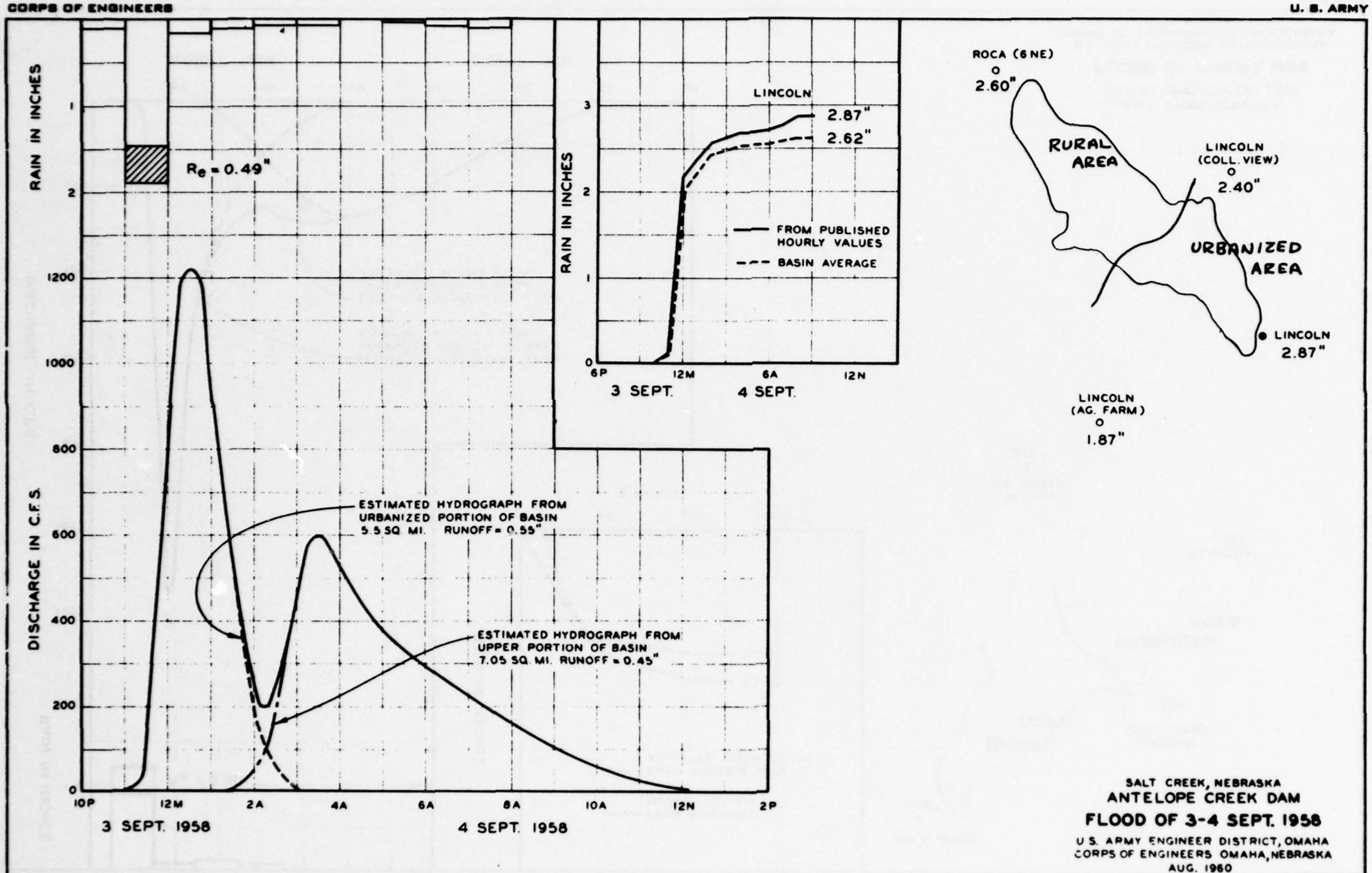


ANTELOPE CREEK BASIN
LINCOLN, NEBR.

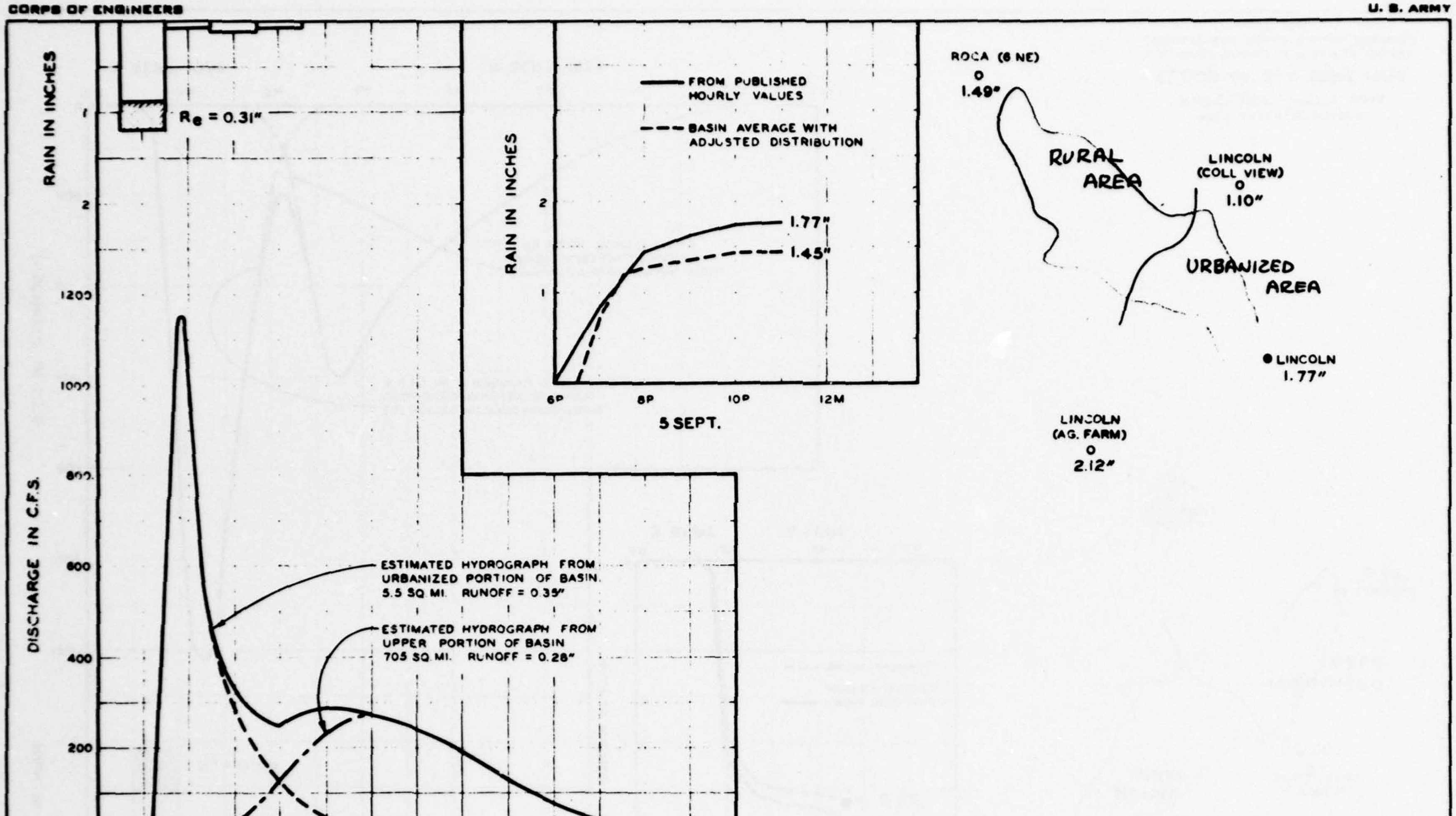
PLATE 1.



SALT CREEK, NEBRASKA
ANTELOPE CREEK DAM
FLOOD OF 24 JULY 1958
U.S. ARMY ENGINEER DISTRICT, OMAHA
CORPS OF ENGINEERS, OMAHA, NEBRASKA
AUG. 1960

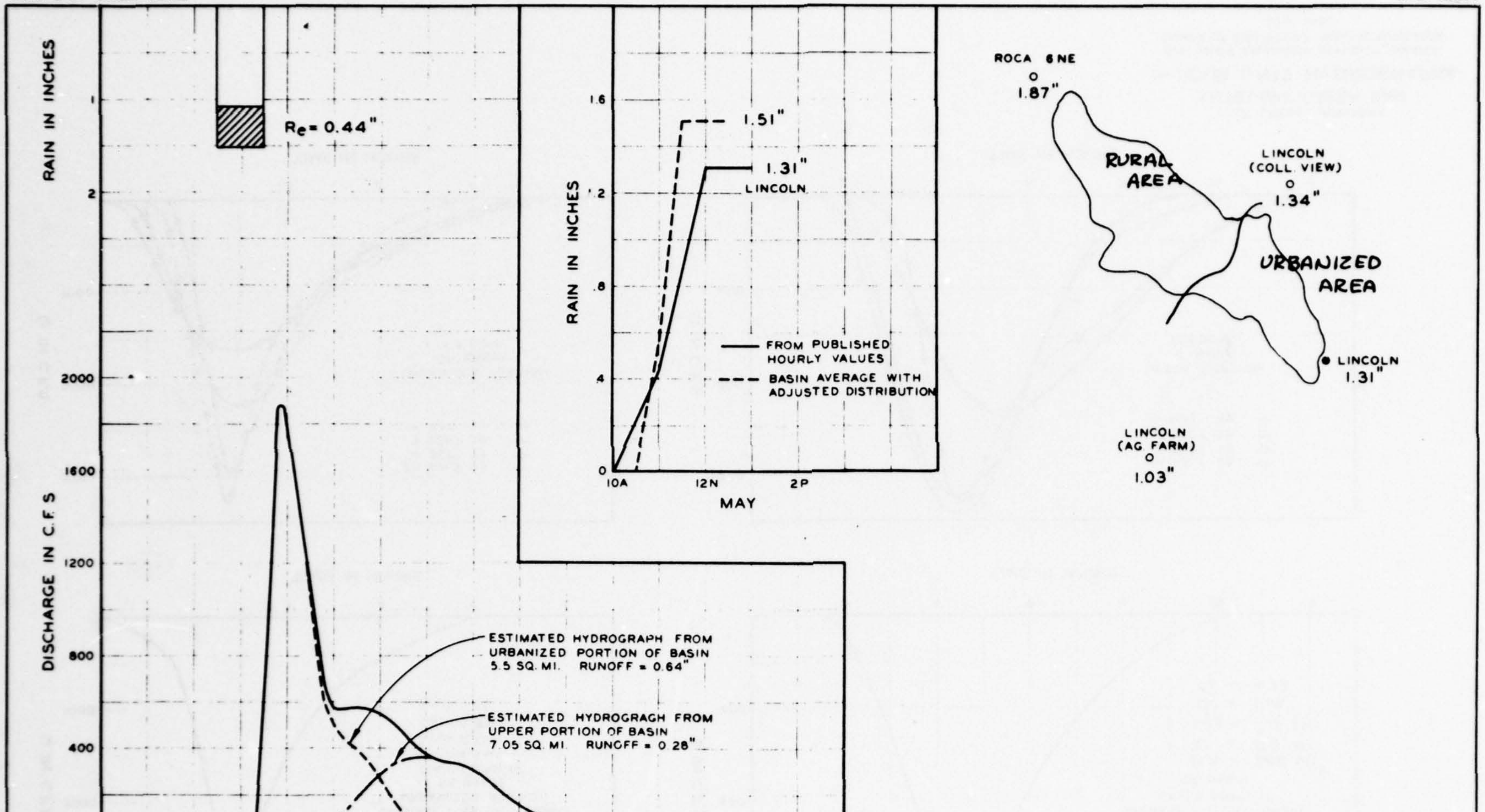


SALT CREEK, NEBRASKA
ANTELOPE CREEK DAM
FLOOD OF 3-4 SEPT. 1958
U.S. ARMY ENGINEER DISTRICT, OMAHA
CORPS OF ENGINEERS OMAHA, NEBRASKA
AUG. 1960



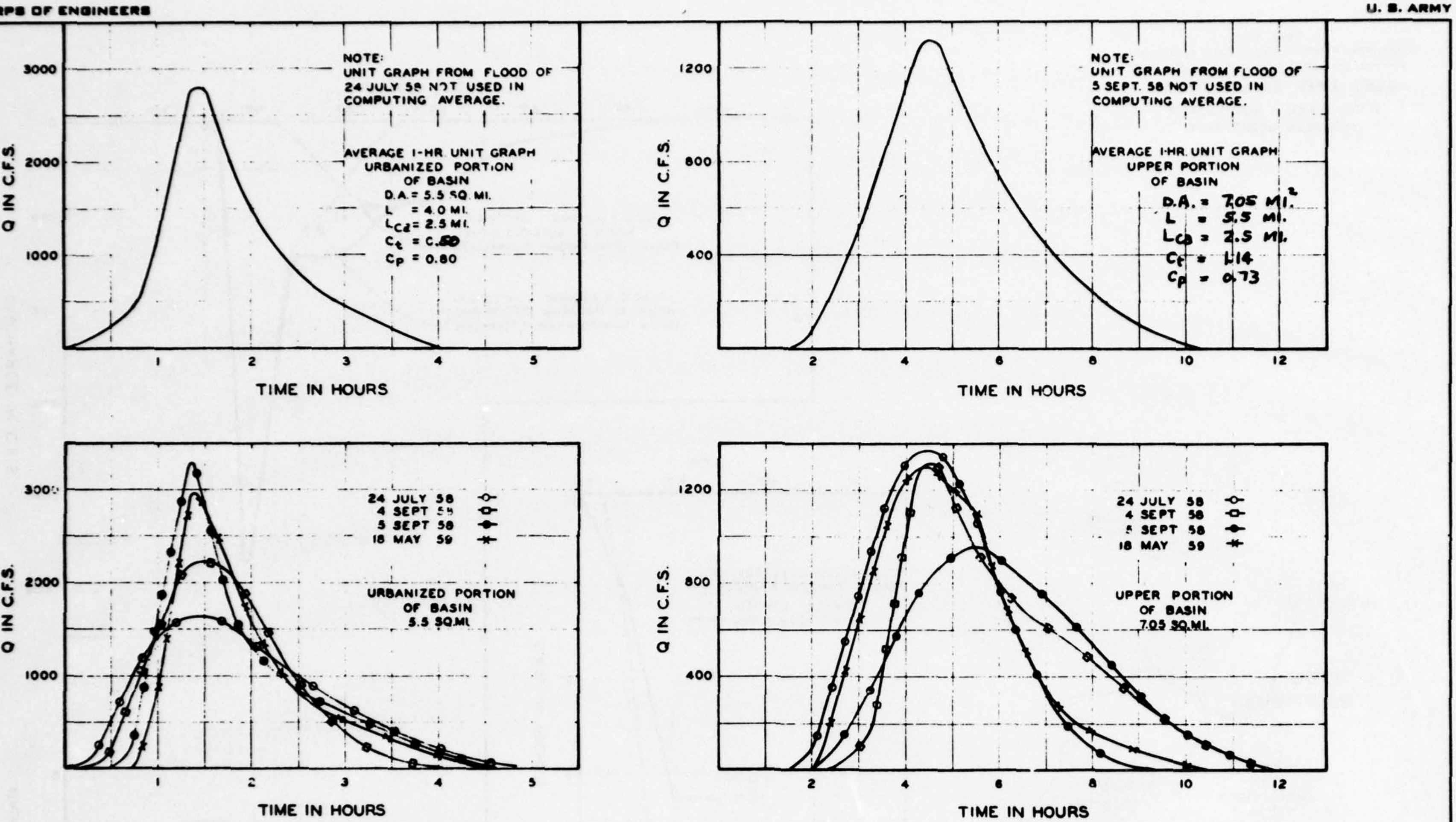
SALT CREEK NEBRASKA
ANTELOPE CREEK DAM
FLOOD OF 5 SEPT. 1958

U.S. ARMY ENGINEER DISTRICT, OMAHA
CORPS OF ENGINEERS OMAHA, NEBRASKA
AUG. 1960

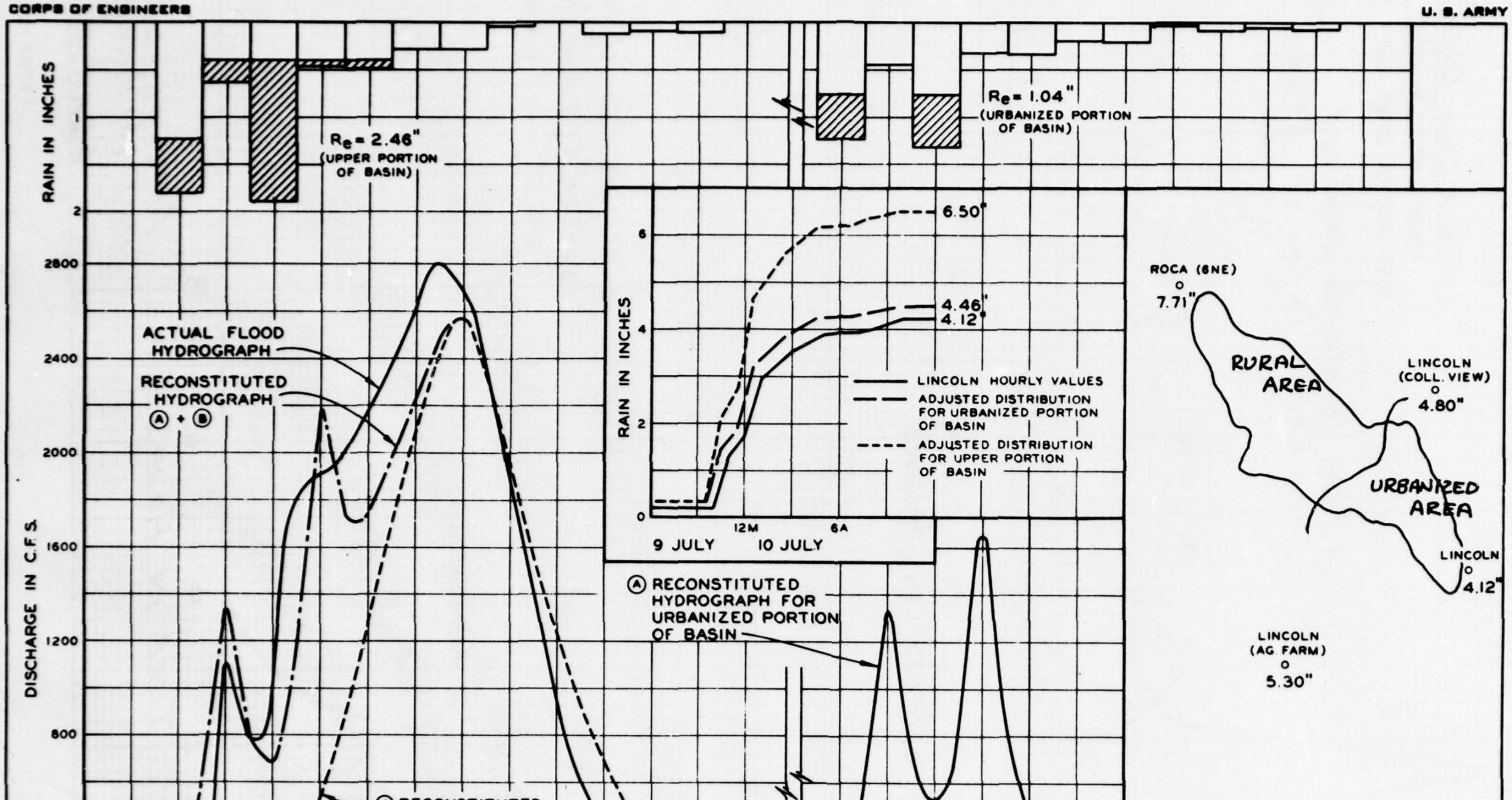


SALT CREEK NEBRASKA
ANTELOPE CREEK DAM
FLOOD OF 18 MAY 1959
U.S. ARMY ENGINEER DISTRICT, OMAHA
CORPS OF ENGINEERS OMAHA, NEBRASKA
AUG. 1960

PLATE 5

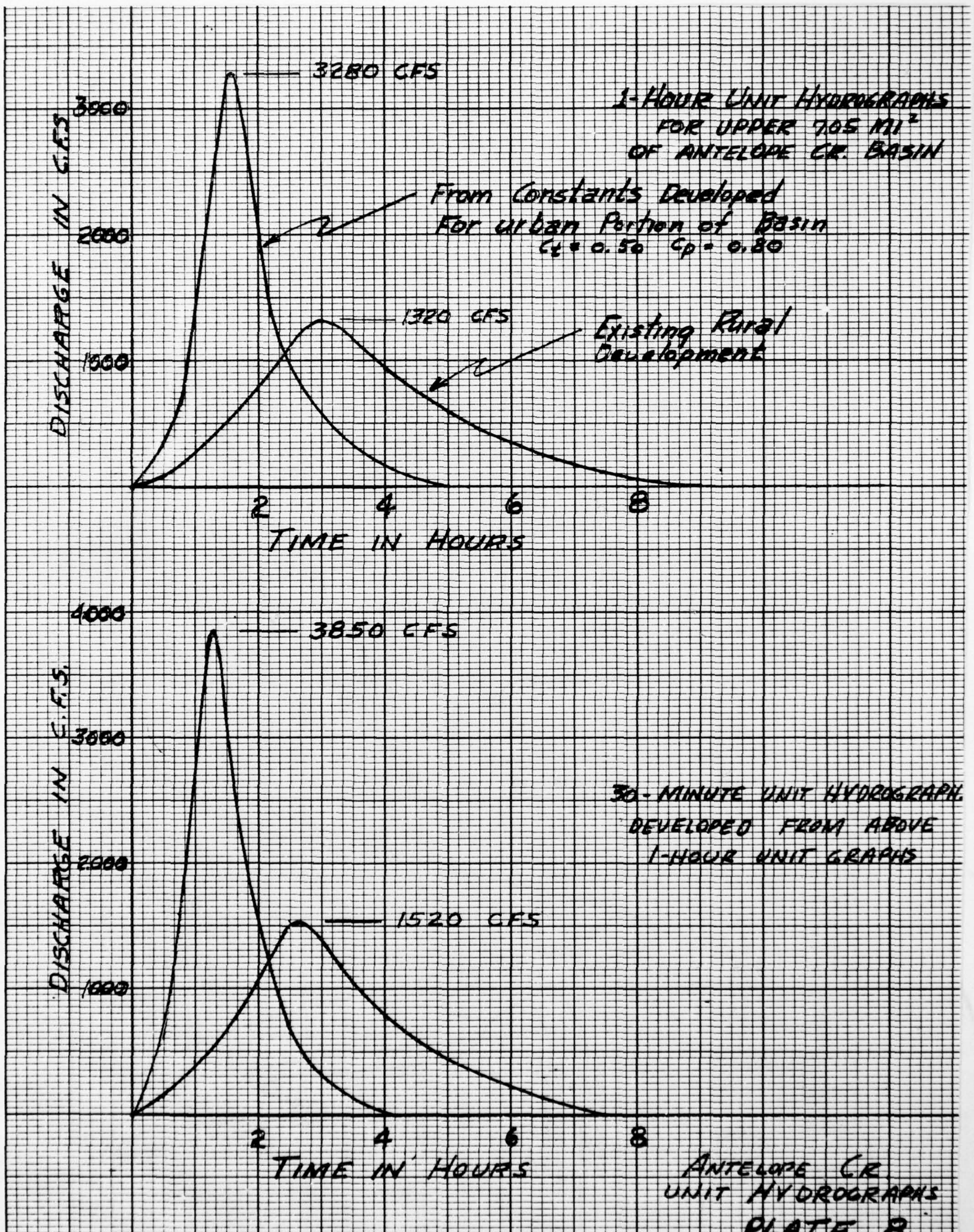


SALT CREEK, NEBRASKA
ANTELOPE CREEK DAM
1-HOUR UNIT HYDROGRAPHS
U.S. ARMY ENGINEER DISTRICT, OMAHA
CORPS OF ENGINEERS OMAHA, NEBRASKA
AUG. 1960



SALT CREEK NEBRASKA
ANTELOPE CREEK DAM 1958
FLOOD OF 9-10 JULY
AT 17TH ST. GAGING STATION
LINCOLN, NEBRASKA

U.S. ARMY ENGINEER DISTRICT, OMAHA
CORPS OF ENGINEERS OMAHA, NEBRASKA
AUG. 1960


3280 CFS
1-HOUR UNIT HYDROGRAPHS
FOR UPPER 7.05 MI²
OF ANTELOPE CR. BASIN
3000
From Constants Developed
For urban Portion of Basin
Ct = 0.50 Cp = 0.80
2000
1320 CFS
Existing Rural
Development
1500
DISCHARGE IN C.F.S.
2
4
6
8
TIME IN HOURS
4000
3850 CFS
3000
30-MINUTE UNIT HYDROGRAPH
DEVELOPED FROM ABOVE
1-HOUR UNIT GRAPHS
2000
1520 CFS
1000
DISCHARGE IN C.F.S.
2
4
6
8
TIME IN HOURS
ANTELOPE CR
UNIT HYDROGRAPHS
PLATE 8

AN ANALYSIS OF THE EFFECTS OF URBANIZATION
ON UNIT HYDROGRAPH CHARACTERISTICS

Discussion

Comment, Mr. Northrop: At least three additional factors will have to be introduced to get good results when we go from natural basins to urbanized conditions:

a. Be sure to adjust for the possible distortion that will be introduced by deriving unit graphs from low frequency floods from urbanized areas and applying them to high frequency design floods.

b. Retention capability of the soil and how it is affected by urbanization (percent of impervious area) is a significant factor particularily when comparing one basin with another.

c. Temporary retention such as bridge restrictions, ponds, depressions, roof storage, etc. should be recognized.

Reply, Mr. K. Johnson: These are pertinent comments that should be recognized in using the unit hydrograph to derive frequency events.

Comment, Mr. Beard: It is interesting to examine the significance of the exponents in the Texas study equations. For urban areas, a basin having stream length about 8 times that of another would have only 2 times the lag. Also, slope has a large influence in speeding flows in natural areas but only a minor influence in urban areas, which is the opposite of general experience.

Reply, Mr. K. Johnson: In visiting with other participants at the seminar they indicated that additional studies made subsequent to the Texas study have resulted in different exponents for the "S" value and for other values in the equation as well.

Question, Mr. W. Johnson: Should the hydrologist responsible for making predictions of future urban runoff spell out the assumptions concerning type of urban development implied in using factors representing imperviousness, storage, storm sewer, etc. How might this be done?

Reply, Mr. K. Johnson: Yes, in projecting future urban development of a presently undeveloped area an attempt should be made to assess the type of development that is likely to occur in the area. With this in mind, an estimate of such factors as percent of imperviousness, storm sewer development, storage, etc. could be made.

The procedure for doing this is not quite clear to me. One approach could be to consult with Planning Commissions who have responsibility for zoning and use their evaluation of how the area will develop.

URBAN HYDROLOGY CONSIDERATIONS
STATE OF HAWAII

by

James B. Clark[1]

Urban hydrology, as associated with urban development in the Hawaiian Islands, has not in the past received special consideration. For interior drainage studies in urban areas, the required rainfall-runoff relationships have generally been extrapolated from those derived during project studies. Basically there are several reasons for this:

a. The order of magnitude of project and urban drainage areas is quite similar, as can be seen in the example presented herein.

b. Being a group of relatively small volcanic islands, rarely are there slopes less than one percent until the immediate coastal area. Because of this, it is seldom necessary to build levees or floodwalls which project above the adjoining ground surface except possibly for freeboard requirements. Ponding depth is minimal and is generally limited to the immediate vicinity of the channels.

c. Project hydrographs have extremely high unit peaks but are of such short duration that coincident peaks are not a problem.

The following is a discussion of a typical example of (urban) hydrology for interior drainage. This analysis was accomplished for Kuliouou Stream, a recently completed Section 205 flood control project, on the island of Oahu.

Kuliouou Valley (plate 1) is on the southeastern corner of the island of Oahu, about 10 miles southeast of Honolulu, on the leeward side of the Koolau Mountains. The drainage area of approximately 1.6 square miles extends about 2.7 miles in length from north to south and about 0.8 mile in width. The lower portion is comparatively flat, while the upper reaches of the basin rise abruptly to 2,390 feet above sea level. The Kuliouou and Honolulu Watershed Forest Reserves occupy about 0.58 square mile of the northern portion of the drainage basin. The Hawaii National Guard Rifle Range, about 0.25 square mile, is in the eastern part of the watershed, and contiguous to the Kuliouou Forest Reserve. The ramaining 0.77 square mile of the drainage area is privately owned and includes the populated flood plain of about

[1]Chief, River Basin Section, Pacific Ocean Division

0.11 square mile. The population of Kuliouou Valley has increased from 800 in 1950 to 1,700 in 1963. The flood plain is presently fully developed. There are no businesses or industries in the valley.

Kuliouou Stream is approximately 3 miles long and intermittently flows into Maunalua Bay, which is very shallow and protected by an offshore coral reef. In the headwaters, there is no well-defined channel and each gulch exists as a very small tributary draining directly into the main stream water course. The average slope of Kuliouou Stream is approximately 620 feet per mile. Slopes in the headwaters area average 1,400 feet per mile, decreasing to 100 feet per mile near the entrance. Tidewater extends about 1,600 feet up the main stream, and throughout the entire length of the main interior drainage ditch, which joins Kuliouou Stream 1,100 feet inland.

Rainfall gradients in the Hawaiian Islands are very steep. Many areas of Hawaii show gradients of 25 inches per mile and more. The most extreme gradient is 118 inches per mile of the 2-1/2 miles from the summit of Mt. Waialeale to Hanalei Tunnel on the island of Kauai. On islands such as Oahu, where mountain peaks do not exceed 5,000 feet, maximum annual rainfall accumulations occur along the ridge lines and decrease with elevation on both the leeward and windward sides. Intensities during storms also follow this pattern but not as pronouncedly as the annual accumulations.

Much of the rainfall in Hawaii results from orographic lifting of the northeast trade winds, the most prominent feature of air circulation in the islands. However, major storms are almost always associated with a migratory low pressure accompanied by widespread heavy rain and southerly winds which produce the "Kona" storms. These Kona storms most frequently occur during the winter months, October through April. All rainfall stations in Hawaii with 50 or more years of record have experienced 24-hour accumulations of at least 8 inches, and the majority have had 12 inches or more. The greatest 24-hour rainfall recorded in Hawaii was 38 inches which fell at the plantation office, Kilauea Sugar Plantation, Kauai, during the storm of 24-25 January 1956. On Oahu, the greatest 24-hour accumulation was 26 inches, recorded at Opaeula which lies on the leeward slope of the Koolau Range, 25 air miles northwest of Kuliouou. On 18 November 1930, a 3-hour rainfall record was established in Moanalua basin located 18 miles northwest of Kuluouou, and the 15.2 inches accumulation is the maximum recorded in the entire state for this duration. For the Kuliouou basin, the average annual rainfall varies from 25 inches at the mouth to 50 inches at the higher elevation.

Kuliouou Basin is divided into three areas, as shown on plate 1, which are significant to the project studies. Area "A" is the 1.2 sq. mi. drainage area above the inlet to the improved channel. Areas

"B" and "C" lie on either side of the flood control channel and include the urban area between area "A" and Kalanianaole Highway. Discharge and stage frequency curves pertinent to the project are shown on plate 2. Since Kuliouou Stream was not gaged, discharge-frequency curves for area "A" and the combined area of A, B, and C (project area) were developed from streamflow data on four nearby streams. The discharge-frequency curve for area "C" was taken directly from the project area on a straight drainage area ratio, the 100-year peak being 3,500 cfs per square mile for both.[1] Derivation of the 50- and 100-year area "C" flood hydrographs from rainfall and unit hydrograph analysis substantiated the area "C" curve. The increased runoff and shorter concentration times in the urban area were offset by a lesser rainfall depth-duration relationship in the lower elevation.

Unit hydrographs for the project area and area "A" were developed synthetically, utilizing mountain lag curves developed by the Los Angeles District and transferred to the Hawaiian Islands on the basis of rainfall and runoff studies.

Unit hydrographs were developed for areas "B" and "C" by transferring the project unit hydrograph to the respective areas using Snyder's synthetic U. H. relations. A unit duration of 10 minutes was used for the project unit hydrograph while a 5-minute duration was used for areas "B" and "C".

Rainfall-loss rates in Hawaii are difficult to determine due to the paucity of adequate rainfall and runoff data. Soils at the higher elevations on the average exhibit a high degree of permeability. The Koolau basalts in the project uplands are very permeable, whereas the deposits in the lower valley exhibit a much lower permeability. For the Kuliouou project, the ground was assumed to be fairly well saturated, and constant loss rates of 1.00 inch per hour and 0.80 inch per hour were used in the mountain and valley areas, respectively. Studies of loss rates in other areas have shown that runoff occurs during periods when rainfall intensity rates are less than the indicated loss rates. Such runoff is attributable to high-intensity rainfall in part of the area and to the imperviousness of part of the area. The loss rate was assumed at 90 percent of the rainfall for the periods having an average rainfall intensity rate of less than the indicated loss rate. The assumption was also made that 50 percent of the valley area would become all-impervious during the economic life of the project. Similarly, in development of the standard project flood for the concentration point at Kalanianaole Highway bridge, 50 percent of the valley area was considered all-impervious and the amounts

[1]This derivation was a peak flow determination, based on an exponential drainage area ratio. Exponents of 0.8 to 1.0 have been found to be satisfactory.

of effective rainfall (total rainfall minus rainfall loss) were computed accordingly.

Due to the construction of flood walls through the urban area, provisions had to be made for the passage of urban runoff into the flood control channel. Area "B" poses no particular problem as it is able, for the most part, to pass over the walls with only a minimum amount of ponding. A small land area immediately upstream of Kalanianiaole Highway, however, will experience some flooding. Existing drainage facilities in this area and throughout the entire area "B" were incorporated into the project without individual detailed studies being done. Area "C", however, contains the major urban area of Kuliouou Valley and did present an interior drainage problem. An existing drainage ditch between Elelupe Street and Kuliouou Stream (plate 1) intercepted runoff from the urban area lying up the valley and transported the flow across the valley, emptying into the stream at a point just above the Kalanianaole Highway bridge. This drainage feature incorporated into the project.

Because of geometric and hydraulic limitations, it was not possible to discharge the 100-year area "C" peak discharge (700 cfs) into Kuliouou Stream; and an analysis of hydrograph volumes was made to determine the adverse effects of temporary ponding. Depth-duration rainfall curves for 50 and 100-year storms were determined for the urban Kuliouou area from Weather Bureau Technical Paper 43, "Rainfall-Frequency Atlas of the Hawaiian Islands." The hyetographs were arranged in critical order and reduced by a 0.05 inch/5 minute loss rate, then combined with the 5 minute area "C" unit hydrograph to produce the hydrographs shown on plate 3. Runoff volume from the 50-year and 100-year floods was computed to be 1,8 and 2.5 inches, respectively.

The design capacity of this interior drainage was limited to 400 cfs. The routed hydrographs for the 50- and 100-year floods are also shown on plate 3, and the resultant stage frequency curve is shown on the insert to plate 2. During the 100-year event, a maximum stage of 4.3 feet (m.s.l.) would flood two houses by 0.3 feet. This was determined to be an acceptable level of damage in view of the cost of alternatives, a pumping station or modification of the bridge over Kuliouou Stream.

The confluence of the interception ditch and Kuliouou Stream was handled in rather a unique way as shown in figure 1. The highway bridge opening is capable of passing the design discharge of 4,900 cfs, with slightly less than desired 2 feet of freeboard. A confluence above the bridge was undesirable because of further encroachment on the freeboard and also because of the fairly high velocity of flow (21 fps) in the main channel. The drain was redirected downstream, sharing a common wall with the channel until it passed to the right of the right bridge abutment, forming a confluence below Kalanianaole Highway. The geometry

of the box culvert through the highway fill limited the drain capacity to 400 cfs.

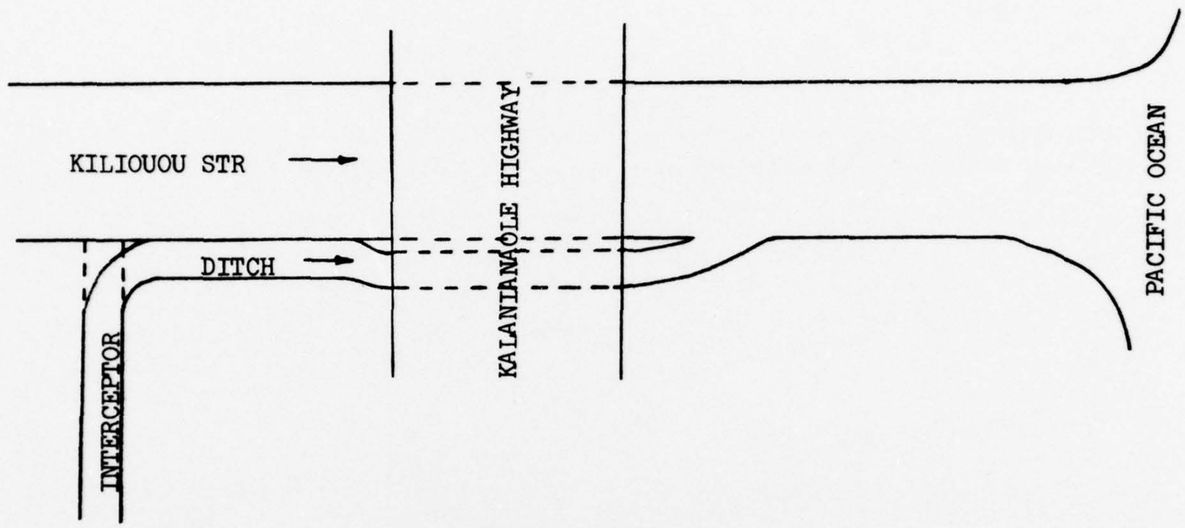
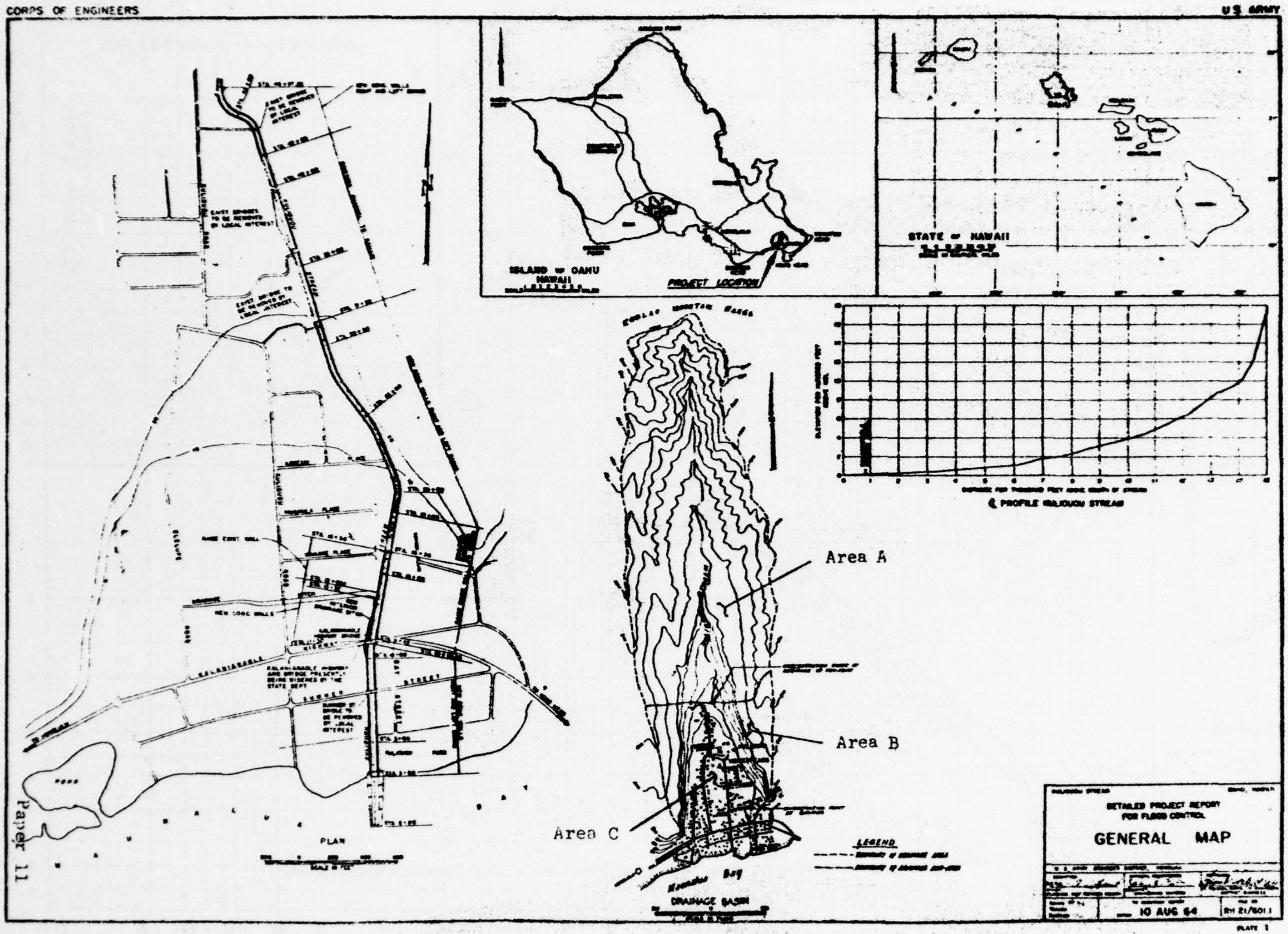


FIGURE 1

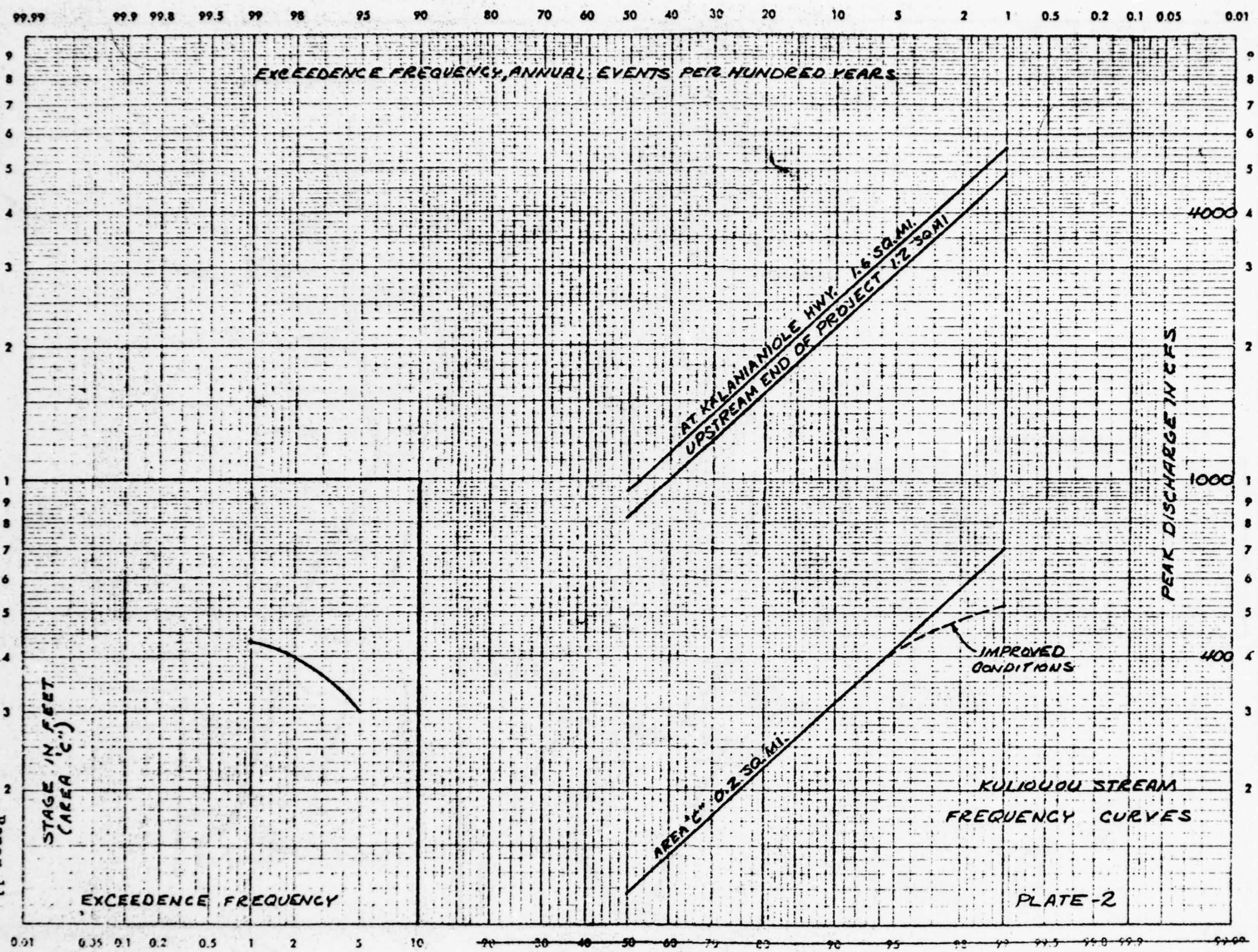
Moving the confluence downstream also benefitted the interior drainage of area "C". The design water surface in the channel is one foot lower (2 ft. m.s.l.) at the downstream confluence and, being below the bridge, eliminates the problem of water backing into area "C" -- unless the bridge becomes plugged with debris and water overtops the channel walls. However, this problem is reduced considerably by a debris barrier constructed by local interests immediately upstream of the lined channel.

In the future, urban hydrology and its effects will have to be more fully considered in the state of Hawaii. Projected urbanization on the relatively lightly developed islands of Maui, Kauai, and Hawaii will generate many problems in the area of effective water control. These problems are in evidence today, particularly on the island of Hawaii, where land development at both high and low elevations, has added impervious surfaces increasing the runoff volume and in addition, interfering with existing natural drainage patterns. In general, there is a lack of well-defined stream channels in the upper and mid-elevation ranges. This coupled with the steep land slopes and high rain intensities results in damaging sheet-type flows which are very prevalent throughout the island chain. For example, the determination and delineation of 100-year events for our recent insurance study for the

city of Hilo, on the island of Hawaii, was made on broad assumptions based on a rainfall-frequency relationship.


CORPS OF ENGINEERS
U.S. ARMY
ISLAND of OAHU
HAWAII
PROJECT LOCATION
STATE of HAWAII
PROFILE KULIOUOU STREAM
KOOLAU MOUNTAIN RANGE
Area A
Area B
Area C
LEGEND
DRAINAGE BASIN
PLAN
KALANIANAOLE
STREET
DETAILED PROJECT REPORT
FOR FLOOD CONTROL
GENERAL MAP
10 AUG 64
PLATE 1

Paper 11



EXCEEDENCE FREQUENCY, ANNUAL EVENTS PER HUNDRED YEARS
AT KALANIANAOLE HWY. 1.6 SQ.MI.
UPSTREAM END OF PROJECT 1.2 SQ.MI.
AREA 'C' 0.2 SQ.MI.
IMPROVED CONDITIONS
PEAK DISCHARGE IN CFS
4000
1000
400
STAGE IN FEET (AREA 'C')
EXCEEDENCE FREQUENCY
KULIOUOU STREAM
FREQUENCY CURVES
PLATE-2

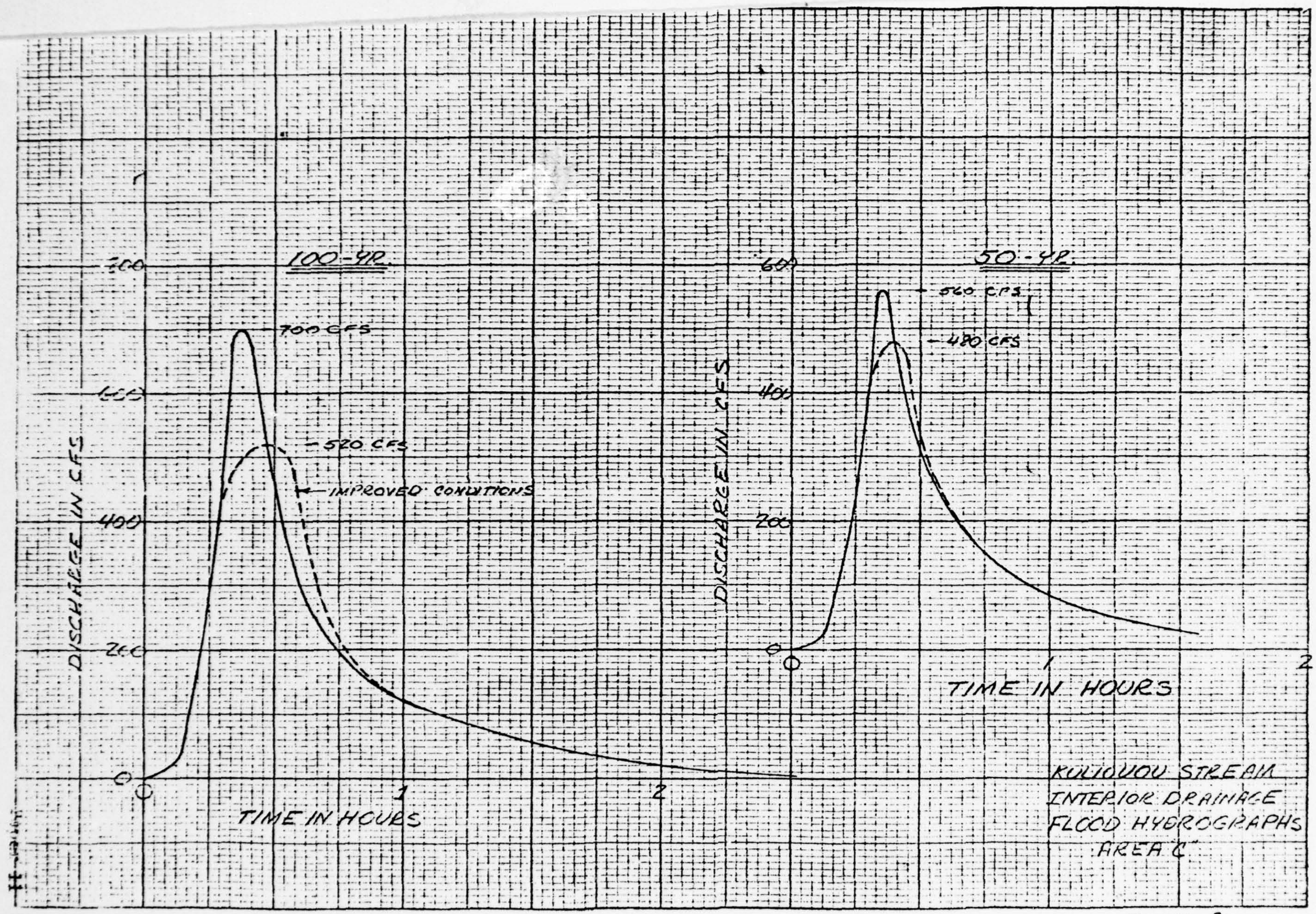

100-YR.
700 CFS
520 CFS
IMPROVED CONDITIONS
DISCHARGE IN CFS
400
200
0
0
1
2
TIME IN HOURS
50-YR.
600
560 CFS
480 CFS
400
200
DISCHARGE IN CFS
0
0
1
2
TIME IN HOURS
KULIOUOU STREAM
INTERIOR DRAINAGE
FLOOD HYDROGRAPHS
AREA "C"

PLATE - 3

KANSAS CITY DISTRICT EXPERIENCES IN URBAN HYDROLOGY

by

William L. Northrop[1]

1. Introduction. During the past two years, the Kansas City District has developed detailed hydrologic criteria for ten small basins with very limited gaging records. These criteria have been used for flood plain reports, technical service type information to local agencies, and survey type studies for flood control measures. The effects of existing and projected urbanization have been factors in a majority of these projects. Also, the effect of temporary ponding induced by man-made restrictions was evaluated in two instances. Since few directly applicable precedents were available and funds were limited, improvisions had to be developed which we think will be of interest. Hydrologic techniques developed in the early studies were utilized and further developed as the program advanced. As a result, hydrologic costs per basin have decreased appreciably.

2. Basin characteristics, soils, and rainfall. These small basins are tributaries to the Missouri River at or near the Kansas Cities. Except for alluvium, the soils have a high content of clay and minimum infiltration rates are less than 0.1 inch per hour. Regardless of tributary size, the overall topographic relief varies from 270 to 435 feet in elevation, the drainage pattern is well developed, channels are deeply eroded and have unusually large capacities at top of high bank. Maximum floods are normally produced

[1] Chief, Hydrology and Hydraulics Section, Kansas City District

by thunderstorms associated with frontal activity and coincidence of Missouri River flooding has to be considered.

3. Urbanization effects. Kansas City District studies have been limited to evaluation of effects on the 50-year, 100-year, and Standard Project Floods. We were forced into separating the effects of increasing channel system efficiency from other urbanization effects on flood peaks. In a few instances the entire basin was occupied by urban type development and the only change was in channel improvement. With the help of guidelines derived from publications, hydrologic data collected during studies, and computations using the latest available techniques, we have established discharge-frequency curves and standard project flood peaks for natural conditions, existing development, and proposed improvements in the drainage system. A few unsuccessful attempts have been made to correlate such factors as slope, drainage area, and peak discharge per square mile for the several basins studied within the Kansas City vicinity. Such a correlation, it was hoped, would yield the necessary peak discharges for flood plain study without expensive individual hydrologic analysis. Apparently additional factors not yet clearly defined have to be evaluated before this can be done satisfactorily.

4. Blue River and Brush Creek. Nearly half of this 272 square mile basin is now urbanized as shown on Exhibit 1. It enters the Missouri River about nine miles below downtown Kansas City, Missouri. Much of the undeveloped land is now held in pasture or unused farmland reserved for future real estate developments. Land surface elevations vary from 1135 ft., m.s.l., in the headwaters to 700 ft., m.s.l., in the Missouri River flood plain

at the mouth. The basin has an overall length of 30 miles and a maximum width of about 12 miles. Within this basin the Brush Creek subarea, drainage area, 29.4 square miles, is completely urbanized with a graded and concrete-lined flowway 3.8 miles long in the lower end. Much of the remaining channel has been improved. Street and storm sewers are generally perpendicular to the channel and there is enough relief to make them effective flowways for major storm runoff. The plan of improvement recommended in a survey report dated April 1968 was accompanied by a detailed hydrologic study. In addition, Flood Plain Information reports have been prepared and published on both the Missouri and Kansas portions of this basin in the past year. Hydrologic studies for the Johnson County, Kansas, portion gave us an opportunity to compare analytically the completely urbanized-improved channel of Brush Creek with the remaining largely undeveloped Blue River tributaries. (Refer to Exhibit 1 for locations, etc.). The steps leading to the isolation of urbanization effects are described below.

a. Discharge-frequency determinations. As a result of the survey report work, discharge-frequency curves based on discharge records at the one long-record gage were available at several locations in the lower Blue River basin. Frequency curves were needed at another six points varving in area from 12 to 30 square miles. After some study, it was decided that available regionalized discharge-frequency methods would give better results than extension of existing records from a drainage area of 182 square miles. Indian Creek, one of the larger tributaries which had a stream gaging station with ten years of somewhat questionable records, was selected as the key point for Flood Plain Report frequency studies since the drainage area above the gage (27 square miles) was representative of

the undeveloped drainage areas. Various methods studied included Beard's Statistical, Kansas Geological Survey, Missouri Geological Survey, and the Comprehensive Framework Study for Region 8. Of the methods studied, the Comprehensive Framework Study (Region 8) was selected as the most appropriate method for all tributaries to be included in the study. In this method the mean discharge and standard deviation computations use the equations shown on Exhibit 2. Comparative frequency curves obtained by the various methods are shown on Exhibit 3. This adopted discharge-frequency curve developed by the framework method was selected largely because it compared surprisingly well with the previous curves developed from gage data at the Bannister Road station (drainage area 182 sq. mi.) and other curves related to this one by peak vs. 0.7 power of the drainage area ratios as shown on Exhibit 4.

b. Adjustment of Brush Creek frequency for urbanization. As previously noted, Brush Creek is completely urbanized and the effectiveness of the channel system has been greatly improved. Since the Comprehensive Framework frequency method represents theoretically rural or undeveloped conditions, some means had to be devised to reflect current conditions. A reanalysis was made of the Brush Creek unit hydrographs using 15 minute ordinates to better define the accelerated peaking due to urbanization. Verification of the unit hydrographs was documented by high-water marks and rainfall measurements. As an aside, it should be noted the engineer making these studies had recently completed two years of university oriented research on the Clark's method of developing flood runoff from rainfall records. The SPF peaks were then computed for various sized drainage areas and plotted producing the curve for Brush Creek shown on Exhibit 5. This

curve was adopted for determination of SPF peak discharges in the Brush Creek basin. The curve is not a total urbanization generalization and is only to be used for the Brush Creek basin. A similar curve constructed for the unimproved streams in the Blue River basin is shown for comparative purposes. Frequency curves for Brush Creek were computed by simple projections of the recomputed SPF peak at the same frequency interval as the natural SPF peak plotting on the comprehensive curve. The remainder of the curve was then drawn parallel to the comprehensive curve. A sample of the adopted frequency curve is illustrated on Exhibit 6.

5. Jersey Creek. The Jersey Creek basin has a drainage area of 5.42 square miles, all fully urbanized and within the city limits of Kansas City, Kansas. Land surfaces vary from 740 feet, m.s.l., in the Missouri River flood plain to 1010 ft., m.s.l., at the drainage divide. The basin is roughly rectangular with a length of 3 miles and a width of 2 miles. The lower two miles of the main stem channel are well incised and have an average fall of 31 feet per mile. Even with a number of restrictive bridges, the main channel has the capacity to pass a 100-year flood with only minor damage. Contrasted to the main channel, several tributaries have only swale drainage and suffer damage from low frequency floods. The flood plain is very narrow varying from 100 feet to 300 feet at standard project level except one short reach that has a maximum width of 800 feet. Jersey Creek passes under the Missouri River flood plain in a conduit. This portion of the Missouri flood plain is included in the Fairfax Levee District. The Jersey Creek location is shown on Exhibit 7.

a. The problem. As a part of an urban renewal project, the City Planning Commission evolved a plan to straighten and pave the Jersey Creek channel and put a fairly long reach in a reinforced concrete conduit. The objective was not so much to increase the channel capacity as to reclaim some land and establish banks that could be maintained in an acceptable manner to fit into the improved neighborhood. Our first study showed that the channel improvement would increase the 100-year and standard project flood peak discharges 25 percent. Since the increase over approximately the natural 100-year discharge would overflow into the Fairfax Levee District and was unacceptable, some alternate had to be developed. The City's consulting engineer wanted to try temporary detention basins to offset the increase of channel improvement and selected five sites for study. We rather reluctantly undertook to evaluate the effectiveness of these detention sites. Actually, the temporary detention storage was much more effective at selected peaks than we had anticipated.

b. Flood of 15 August 1969. Before this study had progressed very far, the maximum 24-hour rain of record occurred at the Kansas City, Missouri, official rainfall station just across the Missouri River from the mouth of Jersey Creek. The storm covered Jersey Creek to an average depth of about 8 inches. An analysis of the hourly records indicated the maximum two hours of the storm had a 75-year frequency. A peak discharge of 5,500 c.f.s. was estimated from the rating curve previously prepared for the conduit at the mouth of Jersey Creek. Only the swale type drainage channels in this basin experienced any recognizable flood damage.

c. Flood discharge-frequency curves. Jersey Creek has no records of flow, dictating the use of generalized frequency determination methods.

Methods examined were: (1) Flood Frequency Relations for Small Streams in Kansas, prepared by the U.S. Geological Survey (Irza) for the State of Kansas; (2) Kansas Streamflow Characteristics, prepared by Kansas Water Resources Board (Ellis and Edelen); and (3) The Comprehensive Framework Study. The first two methods were applied directly to the basin. The Comprehensive Framework Study method was used on the Blue River basin of Missouri and Kansas (Brush Creek) with the coefficients of mean elevation, summer precipitation, C_p, and infiltration index for Jersey Creek inserted. The resultant discharges were scaled down by the ratio of the drainage areas raised to the 0.7 exponential power. The three resultant curves were in general agreement with similar slopes, and the Comprehensive Framework Study gave the lowest discharge for the same frequency. All of these methods are for undeveloped basins. Effects of urbanization and channel improvements had to be applied rather arbitrarily. The reasoning developed in this instance is quoted:

> "Much attention is being devoted nationally to the effects of urbanization and related channel improvement on peak discharges. No specific data have been collected in the Kansas City area, but information at other locations in the United States indicates these effects can be appreciable. After an analysis of all available information, the assumption was made that urbanization would increase the flows of an otherwise rural drainage area by 25 percent and complete straightening and lining the channel would increase this effect to 50 percent. Hence, the base framework curve was increased to 125 percent and 150 percent (as shown on Exhibit 8). One recent study of a convincing nature indicates the percent increase should be proportionately greater for frequencies of peak discharge less than 20-year recurrence."

Later in the study we made some routing computations that demonstrated to our satisfaction that to speed up the flow of water by straightening and

lining the channel and removing some restrictive bridges would bring the peaks of the upper basin and the lower tributaries closer together and could increase the peak more than we had assumed.

d. <u>Detention storage</u>. The purposes of this study are to evaluate the selected sites (see Exhibit 7) for effectiveness in compensating channel improvements, demonstrate the effect of detention storage on 100-year and standard project flood profiles previously computed within the areas affected by backwater, and establish other hydrologic factors which should be recognized in formulation and design of the project. With some modification of the subareas and unit hydrograph previously used for the standard project flood determinations, satisfactory 100-year hydrographs for all inflow points and tributary contributions were developed. Routing studies were made (1) using approximately "road culvert" configuration as outlets and (2) with assumed broadcrested overflow weirs at the 100-year pool elevation. It was concluded that the "road culvert" structure would be much more effective in reducing peak discharges and it was adopted at all sites. It was necessary to determine the required rating of the opening by trial and error routings. The discharge capacity and 100-year pool elevation were established by this study. However, the exact size and shape of openings and the design of the detention structure including the shape of the top was not studied. It was assumed that existing or future street embankments would be modified for this purpose. Computed maximum elevations for the standard project flood could vary somewhat depending on provisions for overflow.

e. <u>Study conclusions</u>. Results of the routing studies are tabulated on Exhibit 9. This shows that a relatively small amount of storage had an unbelievably large reducing effect on the peak discharges. This is

possible only because of the large culvert type outlets that are sized to become effective as detention structures as the outflow approaches the 100-year peak discharge. Although not computed, it was apparent the effect would be proportionately much less on the larger standard project flood peak discharges. This study demonstrated that detention storage at the three selected sites will compensate for the anticipated channel straightening and lining of the main stem Jersey Creek channel as proposed by the consultants. Although the restrictive effect of existing bridges was not utilized to accomplish the results desired, it was recommended that as many as practicable of the restrictions remain to aid in reducing floods in excess of 100-year frequency.

6. Three Mile Creek. This small basin is a right bank tributary of the Missouri River at Leavenworth, Kansas, 31 river miles above Kansas City, Missouri. It drains an area of 6.38 square miles of which the lower 4.5 square miles lie within the urbanized area. The center 80 percent length of the stream has a slope of 40 feet per mile. That portion of the central business district subject to severe flooding is located on a relatively flat flood plain area well above the Missouri River flood plain leve. Channel capacity through this flat area, about 7 blocks in length, is relatively small, 3 to 5 years. A map of the basin is shown on Exhibit 10.

a. Flood frequency curves. A discharge frequency curve for a point near the mouth of Three Mile Creek had been developed from peak flood discharges estimated from a long record of high-water marks and by application of rainfall of various frequencies to the unit hydrograph to determine peak discharges. As a part of the recent study, a frequency curve was computed

by the Comprehensive Framework Study method described previously. Comparison of this curve (see Exhibit 11) with the survey report curve indicated a substantial allowance for urbanization and channel improvement had been induced. This was as it should be for the plan of improvement recommended in the survey report which included a very substantial replacement of restrictive bridges and improved channel.

b. Effect of restrictive bridges on present channel. An inspection of the computed profiles for the 100-year discharge based on the survey report curve adjusted for probability effects indicated two areas of relatively large temporary impoundment. In addition, several large ponds have been constructed in the upper areas that would have effective surcharge during a 100-year flood. These areas are summarized in Exhibit 12. A total of 55 acre-feet of temporary ponding on Jersey Creek reduced the 100-year completely urbanized peak discharge of 7,100 c.f.s. to 5,900 c.f.s. for a 5.4 square mile drainage area. This ponding was distributed in three locations with the most effective area at 10th Street. The rough analysis of ponding in excess of what might be assumed as natural channel storage appears to be 123 acre-feet for Three Mile Creek, which has a total drainage area of 6.38 square miles. Since the basins have similar topography, it would appear that the storage on Three Mile Creek would be about twice as effective as the Jersey Creek storage in reducing peak flows. However, little is known about this type of peak reduction, and funds were unavailable for a routing study, so it was decided to assume only the percent reduction established for the Jersey Creek basin. This reduced the 100-year peak discharge for Three Mile Creek from 7,500 c.f.s. at the mouth to 6,200 c.f.s. This was approximately the 50-year peak that had already been determined and for which the profiles were already computed.

Thus, in recognition of the existing temporary ponding, the 50-year profile as computed in this study was recommended as the Intermediate Regional Flood for existing conditions. Elimination of the temporary ponding or extensive channel improvements would necessitate a readjustment of the Intermediate Regional Flood to the originally computed 100-year peak discharge and profile corresponding to the modified channel conditions.

7. Discussion. The Kansas City District has a backlog of requests for more than 1,000 flood plain reports and processes a continuous flow of technical service requests for 100-year flood levels. Nearly all of these are on small ungaged drainage areas. It was hoped that a careful analysis of several areas in a particular hydrologic province, such as the small tributaries entering the Missouri River in the vicinity of Kansas City, would result in generalizations that could be applied with a fair degree of confidence. As the studies summarized herein have indicated, a number of factors enter that have not been treated in the rather meager guidelines published to date. For instance, the volume of water retained by the soil mantle which is really the factor changed by increased imperviousness is seldom documented. No standardized method has been developed for reporting the efficiency of the main channel tributary system and changes thereto. Little, if anything, has come to my attention on the effect of temporary ponding induced by restrictive bridges, obstructions, or man made improvisions.

9. One of the most perplexing problems we have encountered to date is the effect of urbanization on floods of standard project flood magnitude. We have received very little help to date through published material. Since the relatively shallow soil and high clay content usually encountered in the Kansas City District has led to minimum infiltration rates of less

than 0.1 inch per hour in most cases, it is apparent that reduction in impervious area, one of the principal effects of urbanization, would practically disappear in floods of this magnitude. This has led us to look principally at the increase in efficiency of the drainage system, if any, as the guide to possible effects on peak discharges. As a result, we have adopted both increased peaks and no increase in peaks because of urbanization effect on the standard project flood.

8. Conclusion. We have demonstrated to our satisfaction that much mc research and detailed engineering analysis are necessary before urbanization effects can be reduced to handbook type solutions that will give general acceptable results. At this state of the art, a detailed hydrologic analysis to established flood frequencies and the standard project flood should be made for each basin if time and funds will permit. Results should be coordinated as much as possible in each area with similar hydrologic characteristics to assure reasonably uniform treatment by all Federal, State, local and private engineers doing similar work.

List of Exhibits

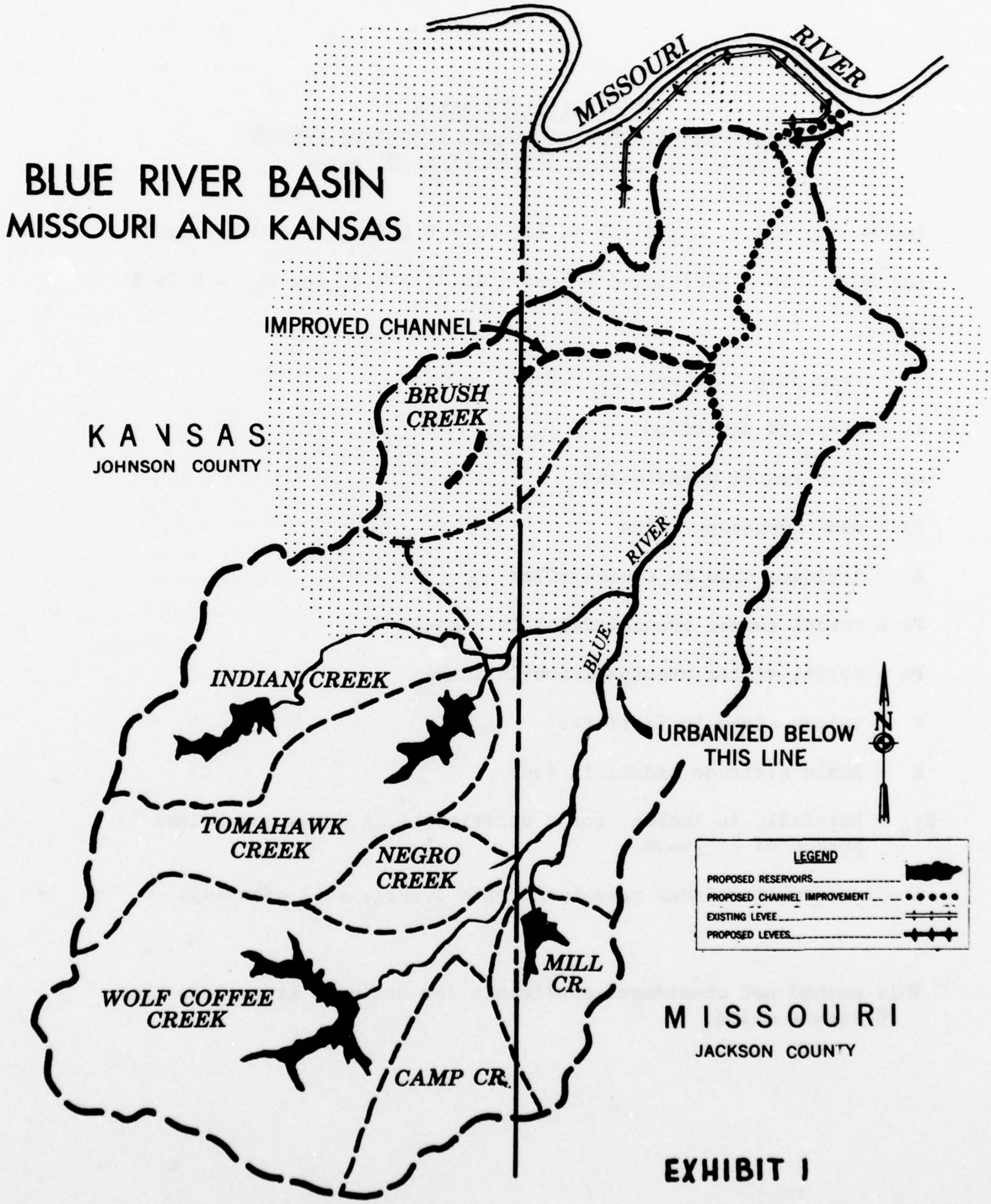

BLUE RIVER BASIN
MISSOURI AND KANSAS
MISSOURI RIVER
IMPROVED CHANNEL
BRUSH CREEK
KANSAS
JOHNSON COUNTY
BLUE RIVER
INDIAN CREEK
URBANIZED BELOW THIS LINE
N
TOMAHAWK CREEK
NEGRO CREEK
LEGEND
PROPOSED RESERVOIRS
PROPOSED CHANNEL IMPROVEMENT
EXISTING LEVEE
PROPOSED LEVEES
MILL CR.
WOLF COFFEE CREEK
MISSOURI
JACKSON COUNTY
CAMP CR.

EXHIBIT 1

MISSOURI RIVER BASIN
Comprehensive Framework Study Frequency Method
Region 8 - Below Kansas City, Missouri

$$\text{Log } M = Cp + 0.74 \text{ Log } A + 2.34 \text{ Log } Ps + 0.29 \text{ Log } Pw + 0.18 \text{ Log } V$$

$$\text{Log } 100S = Fp - 0.03 \text{ Log } A + 3.94 \times 10^{-4} E + 1.38 \text{ Log } R_{24} - 0.04\, I$$

Where

M = mean peak discharge in c.f.s.

S = standard deviation

Cp = peak runoff coefficient

Fp = peak frequency index

A = drainage area in square miles

Ps = normal summer precipitation in inches

Pw = normal winter precipitation in inches

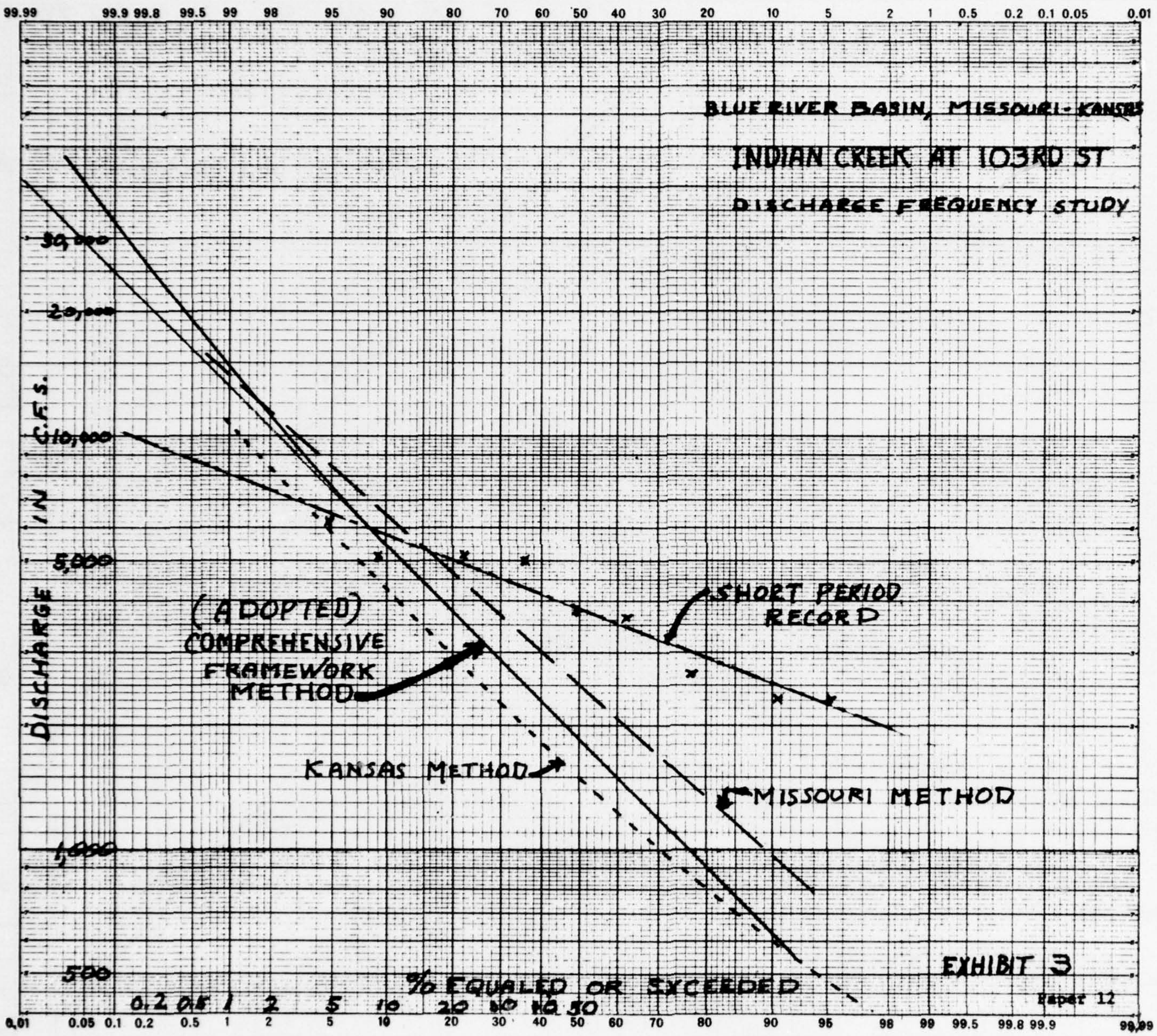
V = valley slope in feet/mile

E = Basin altitude index, in feet

R_{24} = Rainfall, in inches, for a duration of 24 hours and return period of 25 years

I = Infiltration index associated with average soil groupings.

This method not considered applicable for drainage areas less than 20 square miles.



BLUE RIVER BASIN, MISSOURI-KANSAS
INDIAN CREEK AT 103RD ST
DISCHARGE FREQUENCY STUDY
SHORT PERIOD RECORD
(ADOPTED) COMPREHENSIVE FRAMEWORK METHOD
KANSAS METHOD
MISSOURI METHOD
DISCHARGE IN C.F.S.
% EQUALED OR EXCEEDED
30,000
20,000
10,000
5,000
1,000
500
EXHIBIT 3
Paper 12

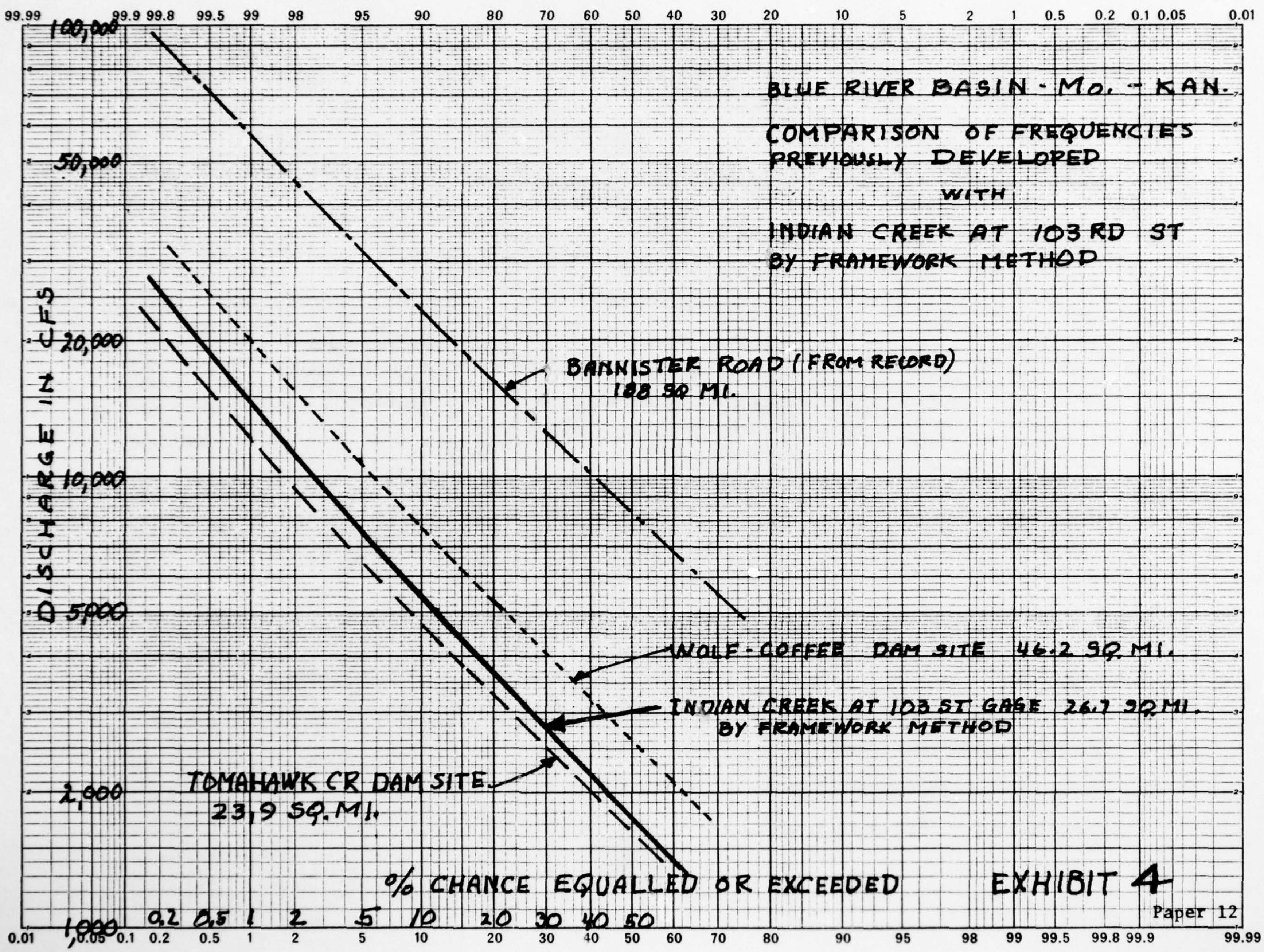


EXHIBIT 4

Paper 12

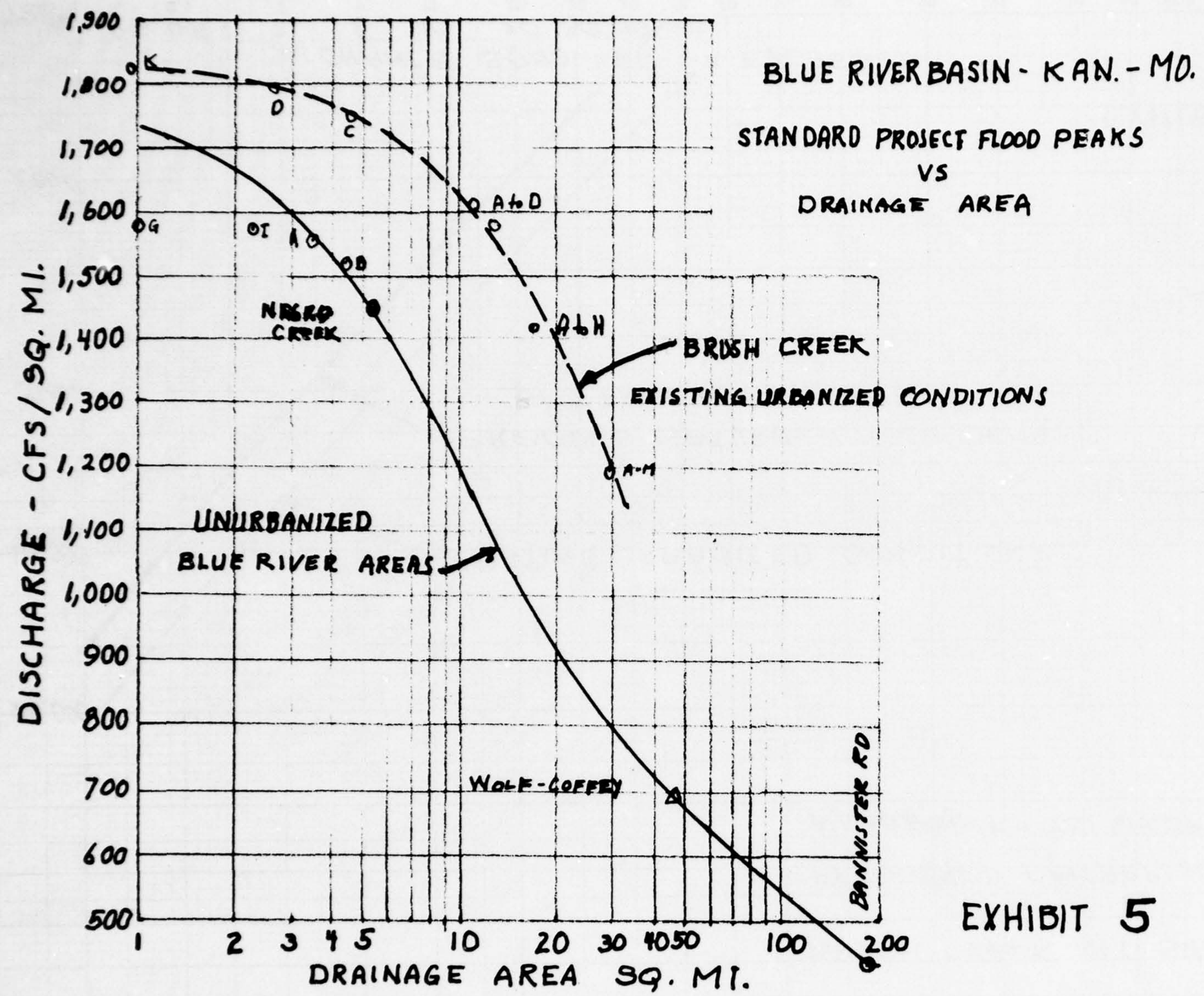
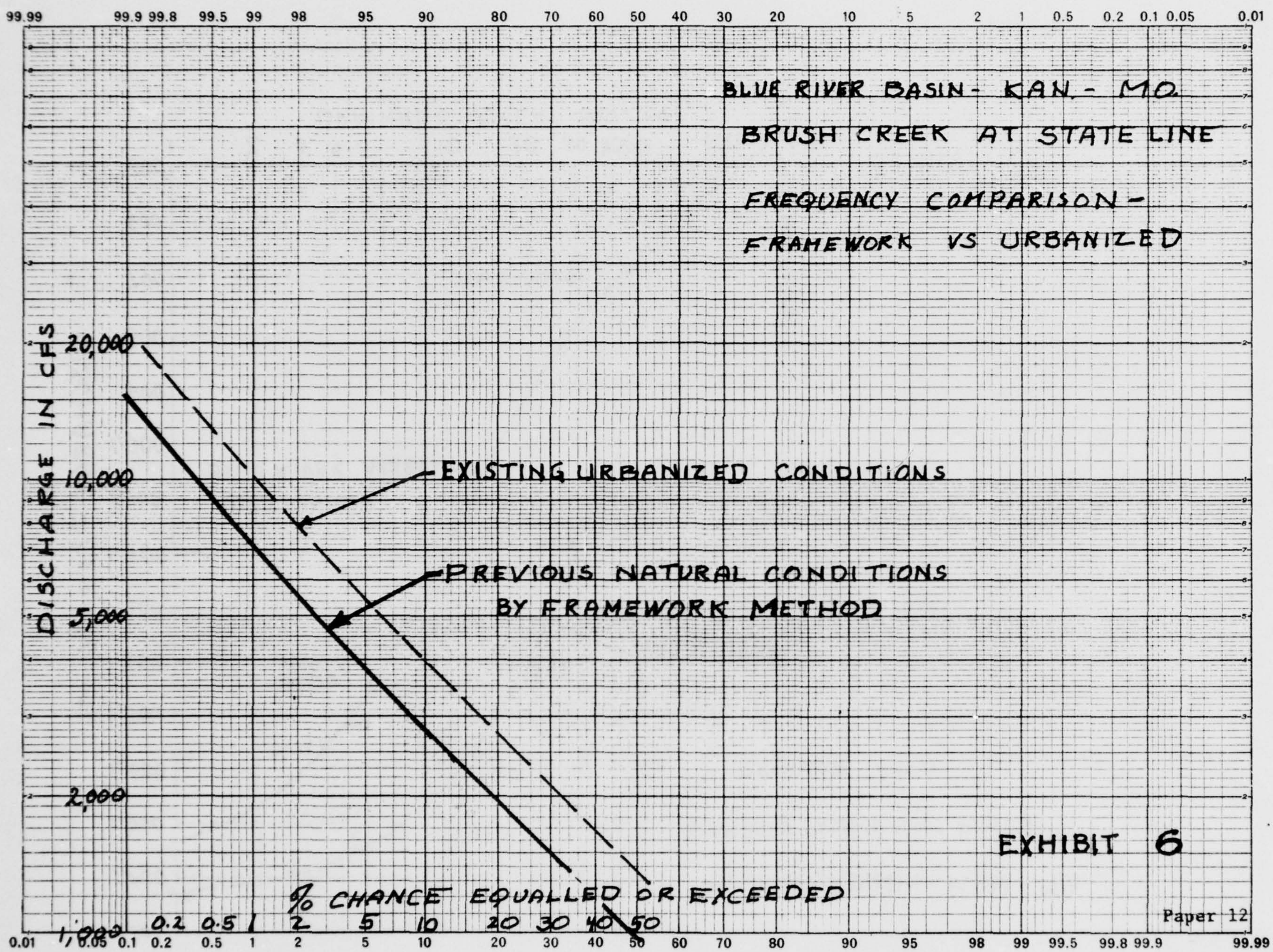
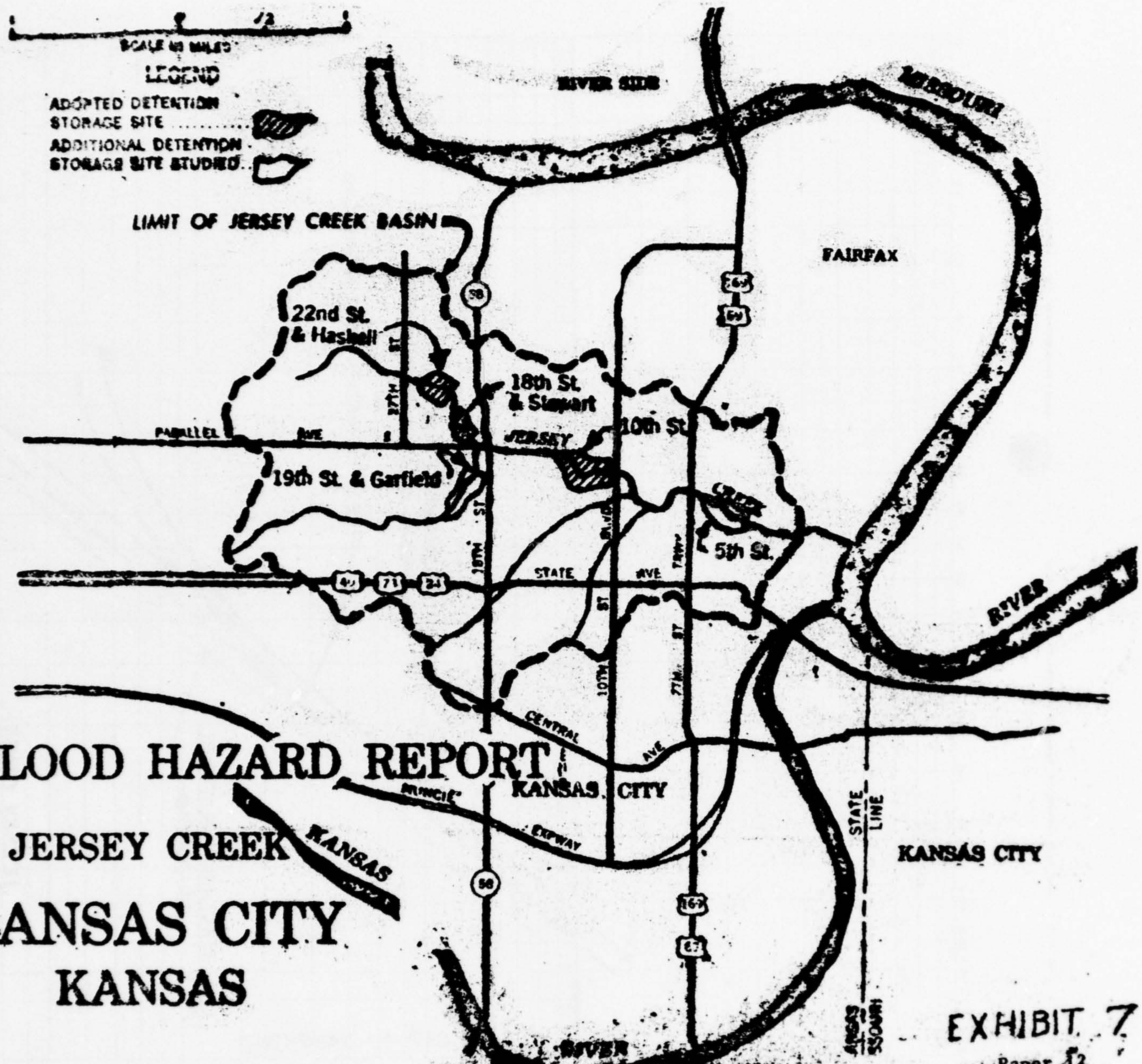

BLUE RIVER BASIN - KAN. - MO.
STANDARD PROJECT FLOOD PEAKS
VS
DRAINAGE AREA
DISCHARGE - CFS / SQ. MI.
DRAINAGE AREA SQ. MI.
1,900
1,800
1,700
1,600
1,500
1,400
1,300
1,200
1,100
1,000
900
800
700
600
500
1
2
3
4
5
10
20
30
40
50
100
200
D
C
A to D
G
I
A
B
NEGRO CREEK
A to H
BRUSH CREEK
EXISTING URBANIZED CONDITIONS
A-M
UNURBANIZED
BLUE RIVER AREAS
WOLF-COFFEY
BANNISTER RD

EXHIBIT 5



BLUE RIVER BASIN - KAN. - MO.
BRUSH CREEK AT STATE LINE
FREQUENCY COMPARISON -
FRAMEWORK VS URBANIZED
EXISTING URBANIZED CONDITIONS
PREVIOUS NATURAL CONDITIONS
BY FRAMEWORK METHOD
DISCHARGE IN CFS
20,000
10,000
5,000
2,000
1,000
% CHANCE EQUALLED OR EXCEEDED
0.2 0.5 1 2 5 10 20 30 40 50

EXHIBIT 6



SPECIAL FLOOD HAZARD REPORT

JERSEY CREEK

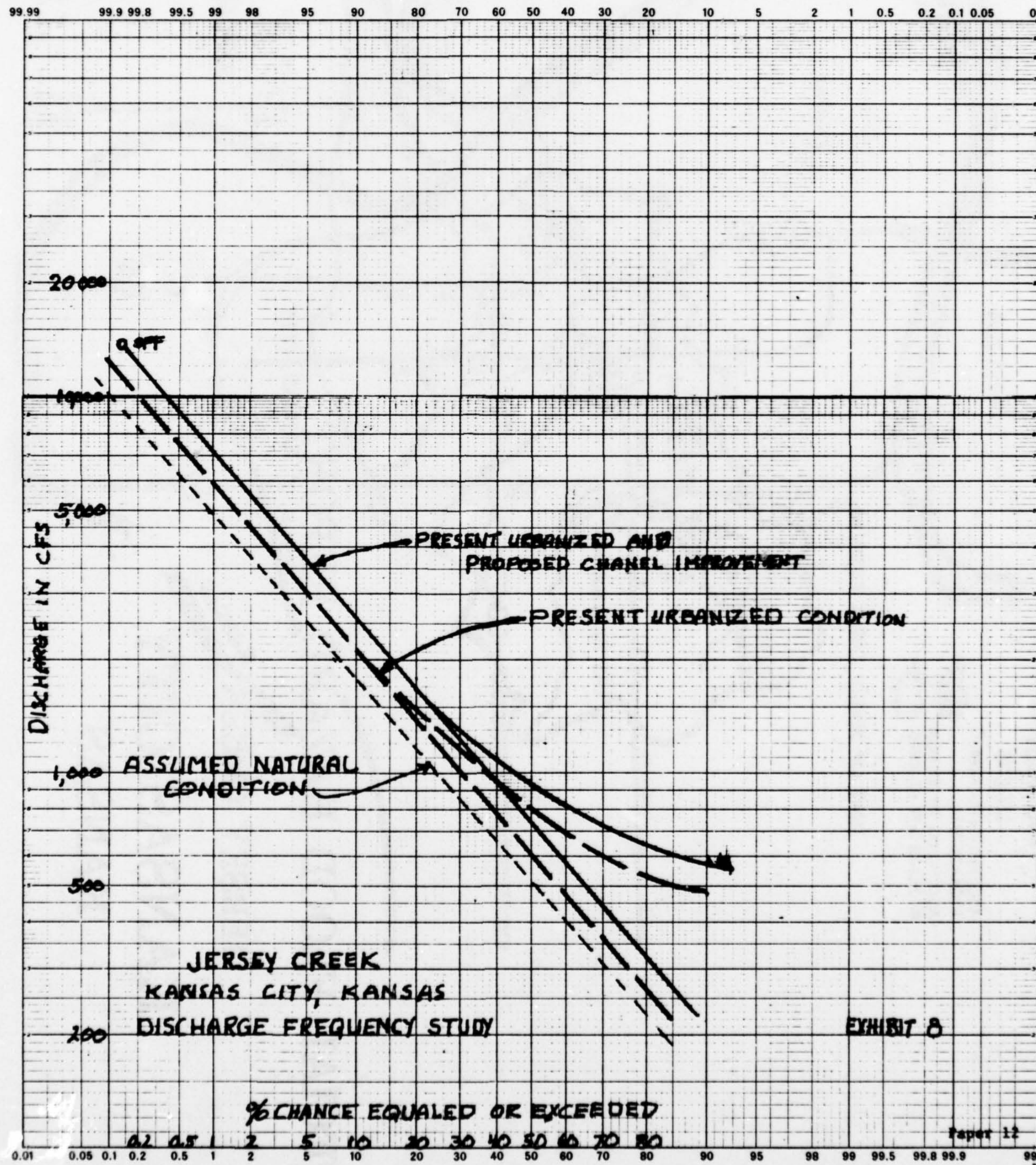
KANSAS CITY

KANSAS

EXHIBIT 7

Paper 12

NO. 3128 LOGARITHMIC PROBABILITY. DESIGNED BY HAZEN, WHIPPLE & FULLER. CODEX BOOK COMPANY, INC. NORWOOD, MASSACHUSETTS. PRINTED IN U.S.A.

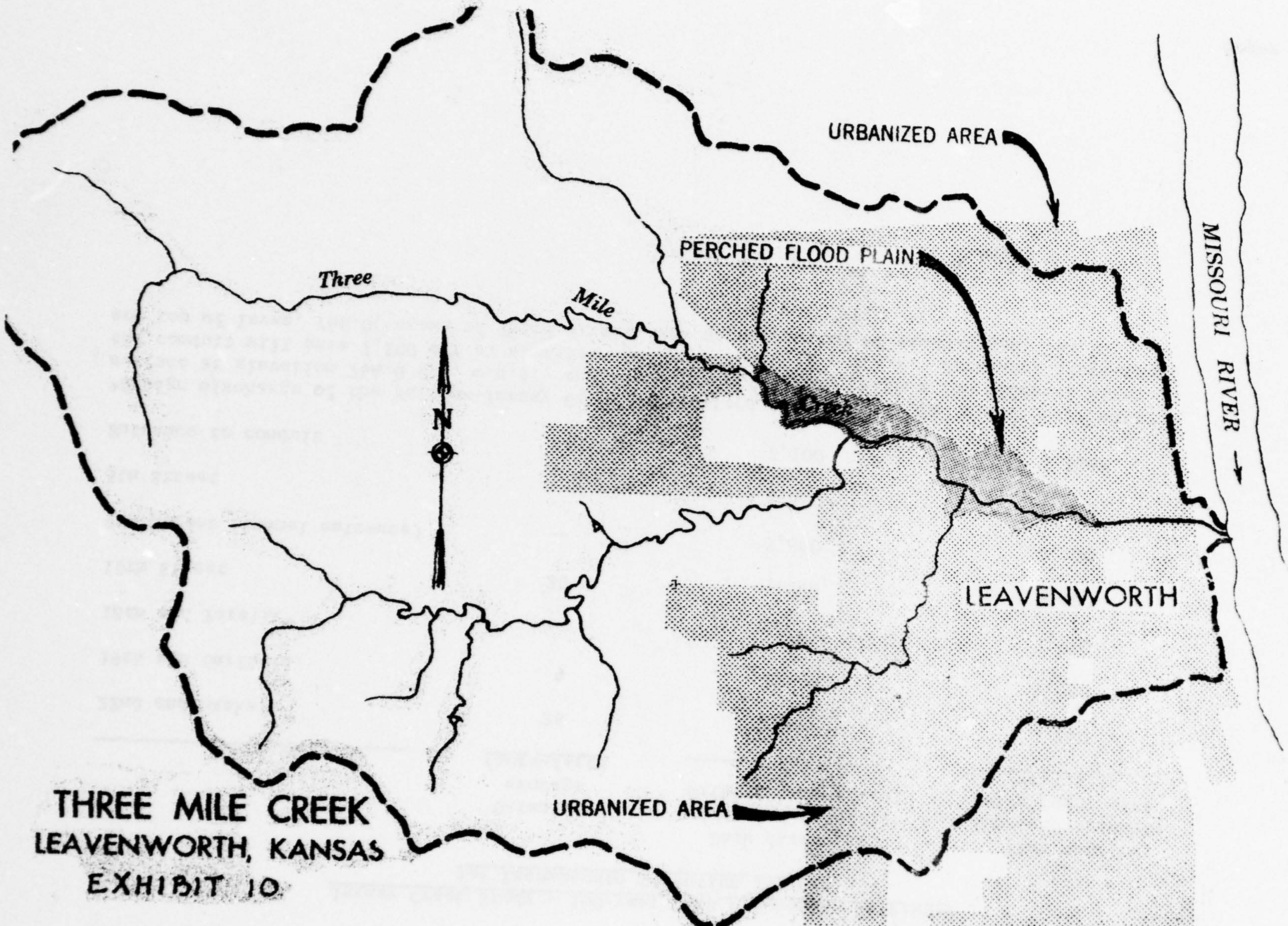


Jersey Creek Study - 100-year Peak Discharges Developed for Recommended Detention Storage

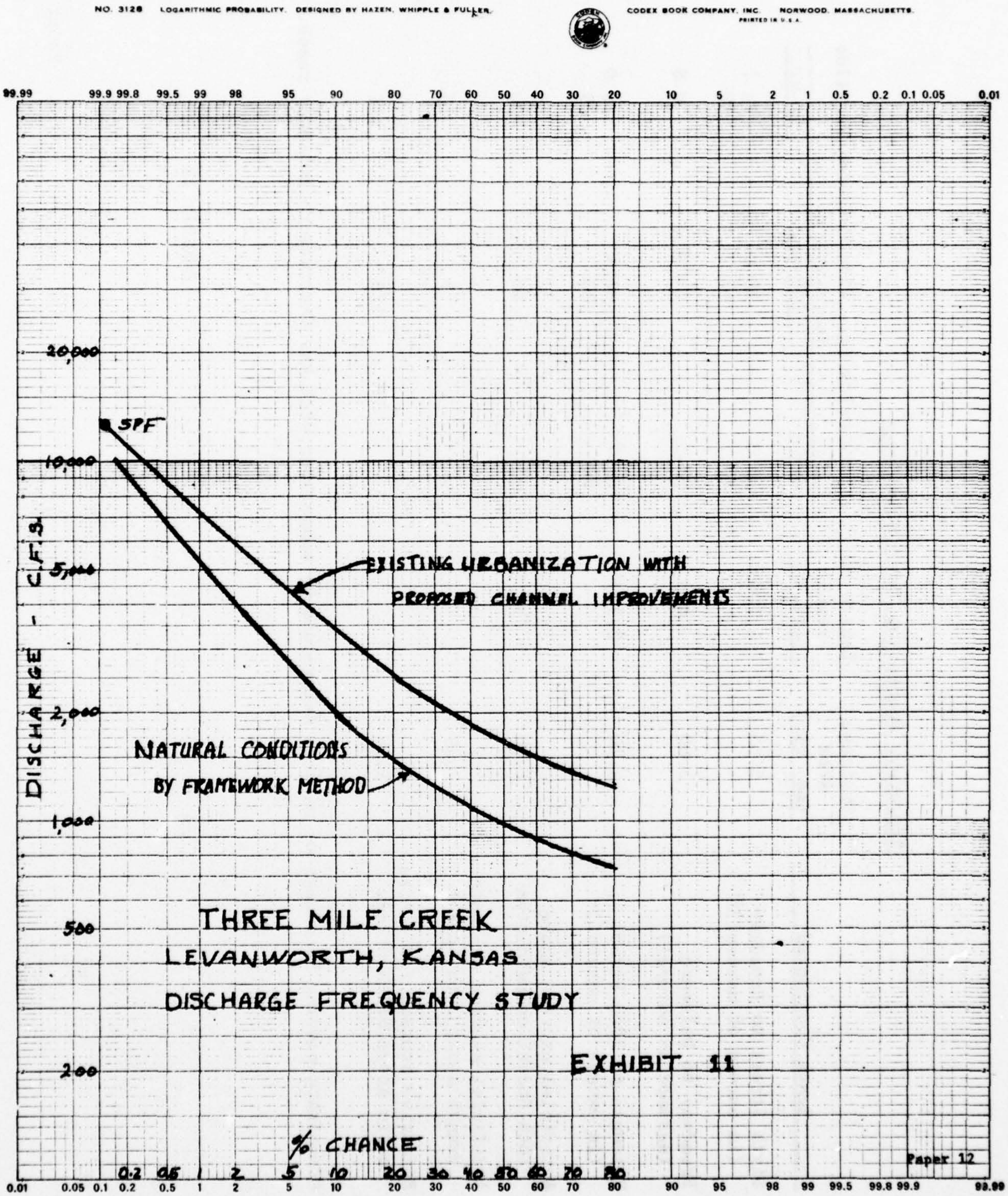
Location	Detention storage (acre-feet)	Peak discharge improved channels without reservoirs (c.f.s.)	Peak outflow improved channels with reservoirs (c.f.s.)
22nd and Haskell	26	1,750	1,240
19th and Garfield	9	1,570	1,050
18th and Parallel	--	3,765	2,970
10th Street	20	4,800	3,400
9th Street (tunnel entrance)	--	5,800	4,400
5th Street	--	6,550	5,150
Entrance to conduit	--	7,100	5,500*

*Design discharge of the Fairfax-Jersey Creek pressurized conduit is 5,900 c.f.s. with water surface at elevation 764.0 ft., m.s.l., and high Missouri River. With a low Missouri River, the conduit will pass 7,100 c.f.s. at inlet elevation 764.0 ft., m.s.l. Pertinent elevations are top of levee, 766.0; crown of Third Street, 764.0; and bottom of sandbag gap, 762.0.

Exhibit 9



THREE MILE CREEK
LEAVENWORTH, KANSAS
EXHIBIT 10



THREE MILE CREEK

LEVANWORTH, KANSAS

DISCHARGE FREQUENCY STUDY

EXHIBIT 11

Paper 12

Temporary Ponding on Three Mile Creek

Reach Considered	Drainage area (sq. mi.)	Length of reach (ft.)	Volume below Initially Computed Profiles (acre-feet)		
			SPF	100-yr.	50-yr.
Sixth Street to Broadway (in excess of natural)*	5.65	1,700	151.3 75	88.8 47	71.7 39
Broadway to Delaware (assumed as natural)	5.80	450	25.3	11.3	8.8
Delaware to 10th Street (in excess of natural)*	4.70	2,350	161.7 30	92.9 33	75.0 29
Artificial storage above 18th Street on main stem	1.00			30	30
Artificial storage on South Branch above 20th Street	.39			13	13
Effective storage for entire basin above Sixth Street	5.65			123	111

*Temporary ponding in excess of natural determined by comparison with the Cherokee to Broadway reach.

Exhibit 12

URBAN HYDROLOGY CONSIDERATIONS IN THE DESIGN OF INTERIOR DRAINAGE FACILITIES FOR LOCAL FLOOD PROTECTION PROJECTS

By

Albert G. Holler[1]

1. General.

Urban hydrology is a subject that is receiving much attention particularly in the United States. It is evident from the literature that a great deal of research funds and efforts are being directed toward a clearer understanding of this subject.

An urban watershed has been described as a watershed in which natural stream channels have been supplemented or replaced by some form of artificial drainage system.1/ This situation is usually accompanied by construction that reduces the infiltration capacity of the area and results in increased rapidity of water flow from its point of impact on the watershed to the outlet of the watershed. Peak flows may be expected to increase from two to four times that of the flow from undeveloped watersheds.2/

The rational method is quite often used to predict a peak runoff for a particular situation. The rational method is summarized by the equation:

$$Q = C\ I\ A$$

in which

- Q is a peak discharge in cubic feet per second
- C is a runoff coefficient
- I is a uniform rate of rainfall intensity in inches per hour for a duration equal to the concentration time of the basin.
- A is the drainage area in acres.

[1]Civil Engineer, Ohio River Division

In utilizing the rational method a number of questions have arisen which reflect the effects of urbanization on the accuracy of the results produced by the method. Among these questions are the following:

1. How valid are the runoff coefficients now in use?

2. Can they sufficiently reflect future (50 years) conditions of the area?

3. How can more accurate values be obtained?

4. How reliable is the rational method?

The choice of a value for the runoff coefficient appears to be the most intangible aspect in the use of the rational method. For a particular drainage area values for measured runoff/rainfall ratios usually vary between wide limits and generally there is a lack of correlation between rainfall characteristics of storms and final values of the ratio of accumulated measured runoff to accumulated rainfall.

Da Costa[3/] has recently proposed new values for the runoff coefficient that may be worthy to note. He derives the runoff coefficient as follows. For a typical hydrograph, let V be the total volume of the hydrograph. Let V_1 and V_2 be the volumes of the hydrograph before and after the moment of peak discharge. Then

$$V = V_1 + V_2 = R$$

where R is the total volume of runoff corresponding to total precipitation P. Let $C_1 = R/P$ = volumetric coefficient of runoff.

The total precipitation P is the product of the intensity of uniform precipitation I acting during time t on a watershed of area A. In equation form,

$$P = I\ t\ A$$

and

$$V = C_1\ I\ t\ A$$

Let t_h be the time corresponding to the formation of volume V_1. The peak discharge of the hydrograph, Q, becomes

$$Q = \frac{a\ V_1}{t_h}$$

in which a = 2 if the ascending part of the hydrograph can be replaced by a straight line without appreciable error.

This equation may be written

$$Q = \frac{a\ V_1}{V}\ \frac{V}{t_h}$$

and

$$Q = \frac{a\ V_1}{V}\ \frac{t}{t_h}\ C_1\ I\ A$$

which has the form of the rational equation in which the runoff coefficient C consists of

$$\frac{a\ V_1}{V}\ \frac{t}{t_h}\ C_1$$

Let $C_2 = a\ V_1\ /V$ and

$C_3 = t\ /t_h$

The runoff coefficient C thus becomes a product of C_2, C_3 and C_1. Factors C_2 and C_3 are not constant but vary with the duration of the rainfall. However, Da Costa claims that for basic hydrographs and an impervious watershed the factors C_2 and C_3 tend to unity. For pervious areas C_2 approaches a value of 0.65 and C_3 approaches a value of 0.7. The coefficients proposed by Da Costa are given in Table 1.

Table 1

Runoff coefficients according to Da Costa[3/]

Factor	Value	Conditions
C_1	0.95	impervious areas
C_2	1.0	impervious areas
	0.65	pervious areas
C_3	0.7	20% or less impervious areas
	0.8	30% impervious areas
	0.9	40% impervious areas
	1.0	50% or greater impervious areas

The values proposed by Da Costa have recently been proposed (April 1970) and it is not known the degree of accuracy involved in randomly applying his coefficients. However, they are significantly different from values that have been used for the runoff coefficient in past situations.

Usually drainage facilities whose capacities have been determined by an application of the rational method will be required to function

over a sufficiently long period of time during which it can be reasonably assumed that runoff relations will be altered somewhat. The runoff coefficient selected must reflect in some way future conditions expected in the drainage area that would affect peak runoff. The future expected peak runoff can then be compared to the design capacity of the drainage facilities to determine their adequacy. Predicting future conditions in a drainage area is subject to much speculation especially for undeveloped isolated areas that depend on particular natural resources or industrial development for their growth. Evidence has been presented in the literature to propose the hypothesis that hydrologically significant impermeable area is related to population density.[4/] From this it may be assumed that it would be possible to determine a value for the runoff coefficient for future conditions from a study of past and present area population and by extrapolating the results into the future. However, the worth of such an approach is subject to the uncertainties usually associated with a population prediction. In a number of cases areas for which drainage facilities have been designed and constructed have experienced a population decline after a growth period. A few examples of this situation for local flood protection projects within the Ohio River Division are presented in Table 2.

Table 2

Population of flood protected areas within the boundaries of the Ohio River Division.

Project	Completed	Population 1950	Population 1960
Huntington, W.Va.	1943	86,353	83,627
Pineville, Ky.	1957	3,890	3,181
Middlesboro, Ky.	1939	14,482	12,607
Bradford, Pa.	1961	17,354	15,061
Elkins, W.Va.	1949	9,121	8,307
Johnstown, Pa.	1943	63,232	53,949
Wellsville, O.	1942	7,854	7,117
Punxsutawney, Pa.	1950	8,969	8,805

Although much research is being directed toward a clearer understanding of the effects of urbanization on runoff relationships, very few conclusions and new design criteria have been offered. Therefore, design of drainage facilities is proceeding by methods that have long been in use. The consequence of utilizing such traditional methods for drainage design in areas where future development is expected is likely to be a reduction in degree of protection as urbanization proceeds.

It is expected that any future information on urban hydrology will most likely be utilized by the Ohio River Division in the design of interior drainage facilities for local flood protection

projects. To determine what additional steps could be taken to include the effects of urbanization on interior drainage design, two local protection projects are briefly described. These are:

1. A levee in two sections for the town of Middlesboro, Kentucky on Yellow Creek.

2. An earth levee and two pump stations for Dayton, Kentucky, located on the left bank of the Ohio River across from Cincinnati. It is hoped that by presenting the basic design of the interior drainage facilities for these projects the effects of urbanization on the hydraulic design of these projects may be determined and improved design methods suggested.

II. Middlesboro, Kentucky.

Middlesboro is located in the southeastern corner of Kentucky as shown on Figure 1. It lies within a relatively level valley 6 square miles in area in the upper portion of the Yellow Creek watershed. Yellow Creek itself is a tributary of the Cumberland River and enters that stream about 600 miles above its mouth. The terrain surrounding the basin is quite rugged with elevations over 2000 feet above the valley floor. The area above the town is generally covered with second-growth timber and is cleared only to permit minor farming operations or for use as sites for coal mining settlements. The town was built in 1890 and enjoyed rapid growth until it encountered financial difficulties and collapsed when the

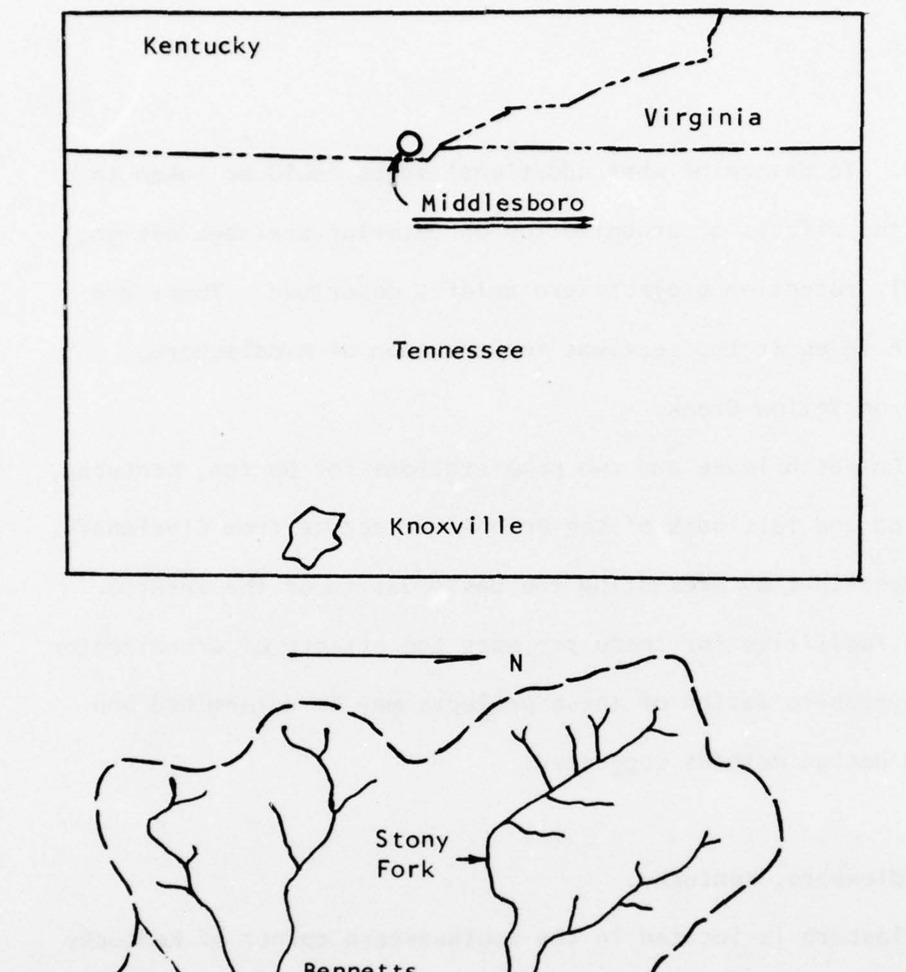


Fig. 1. Middlesboro, Kentucky, vicinity map and drainage basin plan.

ore beds which were to have made it an iron and steel producing center were found to be inadequate. Since the collapse in 1893 the town has gradually developed into a stable community with a population in 1940 of nearly 12,000.

Middlesboro had been subject to recurrent floods caused primarily by inadequate discharge capacity in Yellow Creek below the town. As part of the original development of the town the alignment of Yellow Creek was changed by the construction of a canal through the main part of town. However, the canal proved to be insufficient to accommodate the rapidly concentrated flood flows and later the Federal Government was called on to construct a flood protection project and on 15 September 1940 a waterway four miles long which collected and diverted floodwaters was completed. Altogether the runoff from approximately 38 square miles was diverted around the city. The diversion works consisted of channels, levees, and appurtenant structures. The town was protected by a levee $1\frac{1}{2}$ miles long. The diversion works were designed to carry the probable maximum flood with ample freeboard on the levees. The restricted outflow from the basin, however, still subjected the city to backwater floods and levee extensions were proposed to alleviate this situation. The area subject to flooding consists largely of residential, commercial, and industrial developments with some undeveloped areas.

The proposed improvement consists of an addition to the existing flood protection project by the construction of a supplemental levee system. The new works consist of two main sections: the tannery section and the left bank section. The tannery section, which consists of two short levees connected by a small knoll will be joined to the lower end of the existing diversion levee and will extend upstream approximately 1500 feet along the left bank of Yellow Creek where it ties into a railroad embankment. The top of the levee will be at the same elevation as the existing levee.

The left bank section starts at a knoll about 1500 feet upstream from the upstream end of the tannery section and extends upstream along the left banks of Yellow and Little Yellow Creeks where it ties into high ground.

Total drainage area inclosed by the diversion works and proposed levees is 3600 acres (5.6 sq mi). Approximately $\frac{1}{4}$ of this area, located just south of Middlesboro, has steep slopes that are covered with trees and vegetation. The remaining area lies entirely within the city limits and is relatively flat. Only a small part of the area is paved or covered with roof surface most of it being in lawn and pasture coverage. The 1946 storm water sewer system of Middlesboro was not extensive and was inadequate during heavy storms. This condition together with flat surface slopes resulted in a relatively slow runoff rate.

The total area is broken up into five natural drainage areas which are shown in Figure 2. Gated outlets are provided for each

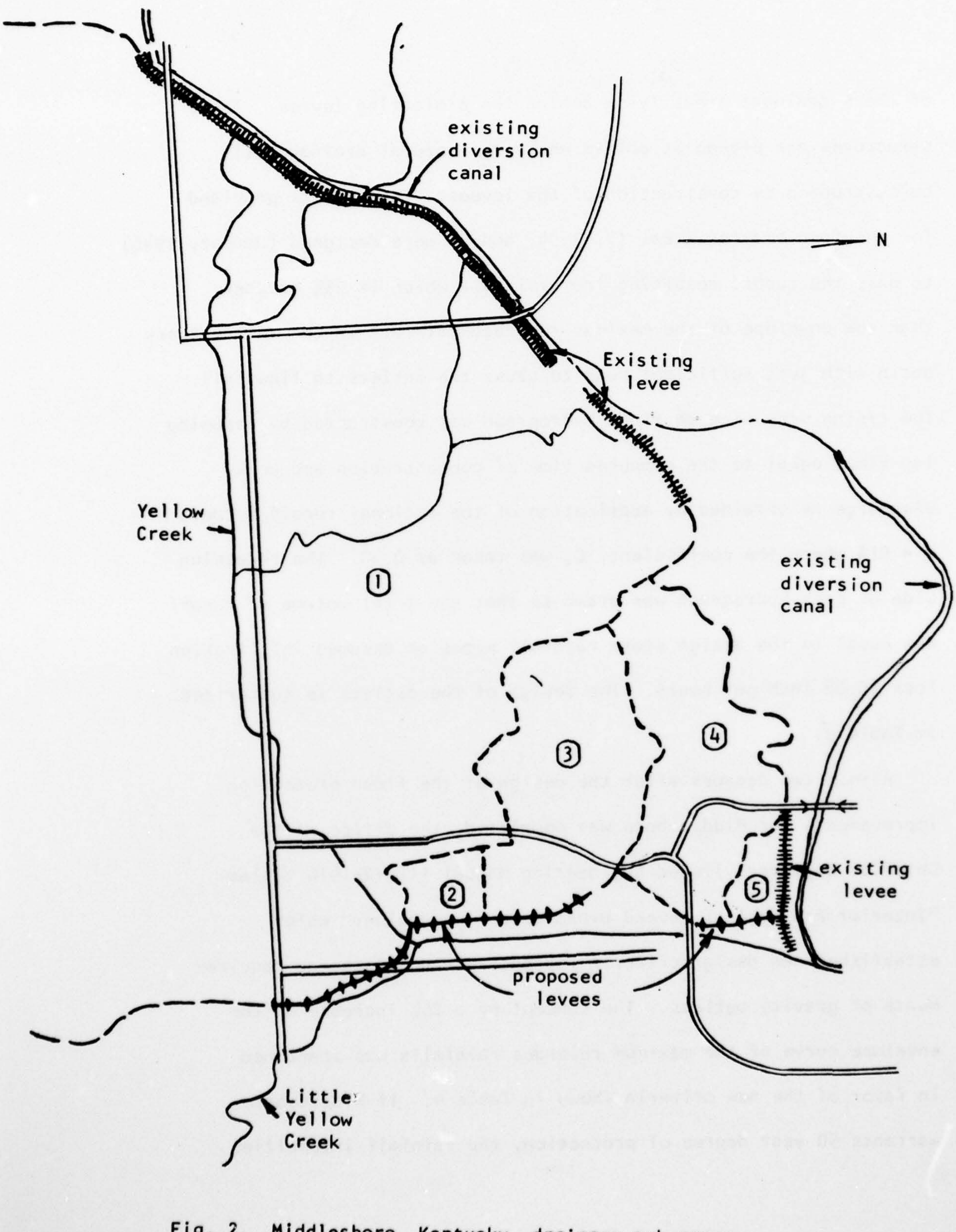


Fig. 2. Middlesboro, Kentucky, drainage sub-areas.

of the 5 drainage areas lying behind the protecting levees. These structures are placed at points where the natural drainage will be obstructed by construction of the levees. The outlets provided for the four smaller areas (2, 3, 4, and 5) were designed (January 1946) to pass the runoff resulting from rainfall which is 25% greater than the envelope of the maximum recorded rainfall in the Yellow Creek basin with just sufficient head to cause the outlets to flow full. The rising side of each flood hydrograph was constructed by assuming lag times equal to the computed time of concentration and peak discharge as obtained by application of the rational runoff formula $Q = CIA$ where the coefficient, C, was taken as 0.40. The recession side of each hydrograph was drawn so that the total volume of runoff was equal to the design storm rainfall minus an assumed infiltration loss (0.08 inch per hour). The design of the outlets is summarized in Table 3.

Almost two decades after the design of the flood protection improvements for Middlesboro was completed, the Office of the Chief of Engineers issued Engineering Manual 1110-2-1410 titled "Interior Drainage of Leveed Urban Areas: Hydrology" which established new design criteria for determining discharge requirements of gravity outlets. The concept of a 25% increase on the envelope curve of the maximum recorded rainfalls was abandoned in favor of the new criteria shown in Table 4. If Middlesboro warrants 50 year degree of protection, the rainfall intensities

Table 3

Summary of data used in design of interior drainage outlets for Middlesboro, Kentucky local flood protection project.

Section	Area (Acres)	Time of Concentration (Minutes)	Rainfall Intensity (In/Hr)	Runoff Coefficient (Percent)	Peak Flow (CFS)	Required Culvert
1	3213	240	1.23	0.40	1590	3 - 8'x7'
2	26	40	4.27	0.40	45	24'' Ø
3	170	90	2.56	0.40	174	42'' D
4	173	90	2.56	0.40	177	30'' D
5	18	35	4.70	0.40	34	18'' D

Table 4

Required degree of protection for interior drainage facilities of local flood protection projects as recommended in EM 1110-2-1410

Ponding Stage Design Objective	Description of Damage or Limiting Factor	Return Interval of Degree of Protection Desired in Years		
		Class I	Class II	Class III
A	Minor Adverse Effects	2	1.5	1
B	Intermediate Design Objective	10	5	3
C	Normal Design Limit	100	50	25
D	Critically Severe	SPS	SPS	SPS

Class	Description
I	Concentrated commercial and industrial section
II	Highly developed residential-commercial section
III	Relatively low-valued urban section

for a duration equal to the reported time of concentration are as shown in Table 5. A new runoff coefficient may be calculated based on the peak inflow given in Table 3, the area of each section and the 50 year rainfall intensities. This new runoff coefficient is larger than the original 0.4 value used and represents some degree of urbanization which the areas may undergo before the degree of protection is lowered.

Table 5

Runoff coefficients based on 50 year storm and existing culvert capacities.

Section	Rainfall Intensity 50 Year Storm In/Hr	$C' = \frac{Q}{AI'}$
1	0.94	0.53
2	3.74	0.47
3	2.14	0.47
4	2.14	0.48
5	4.03	0.47

If the method proposed by Da Costa is used to establish a runoff coefficient the value which results is

$$C = C_1 \times C_2 \times C_3$$

$$= 0.95(0.4) \times \left[1(0.4) + 0.65(0.6)\right] \times 0.90$$

$$= 0.38 \times 0.79 \times 0.9$$

$$C = 0.27$$

and is significantly below the value of 0.4 used. If the value for the runoff coefficient of 0.27 is used, the sizes of the required culverts are reduced.

III. Dayton, Kentucky.

Dayton is an incorporated city in northern Kentucky. A location map is presented in Figure 3. The 1960 population was 9050. Dayton is bounded upstream by an unincorporated area in the Ohio River flood plain and downstream by the City of Bellevue. The study area is approximately 240 acres of light industrial, commercial, and residential development situated on the flood plain between the Ohio River and steep hills. Dayton has recently annexed 800 acres on top of the southern bounding hills to provide for additional residential and light commercial growth. Potential expansion for Dayton is now limited to the Ohio River flood plain upstream from the existing corporate area and smaller areas within the corporate limits where urban renewal development is suitable. Dayton is important as a light manufacturing commercial and residential area within the larger Cincinnati metropolitan center. The population trend in Dayton showed a slight increase between the 1950 and 1960 census (1950 - 8977; 1960 - 9050). Present indications are that substantial population increases will be experienced during the 1960 decade.

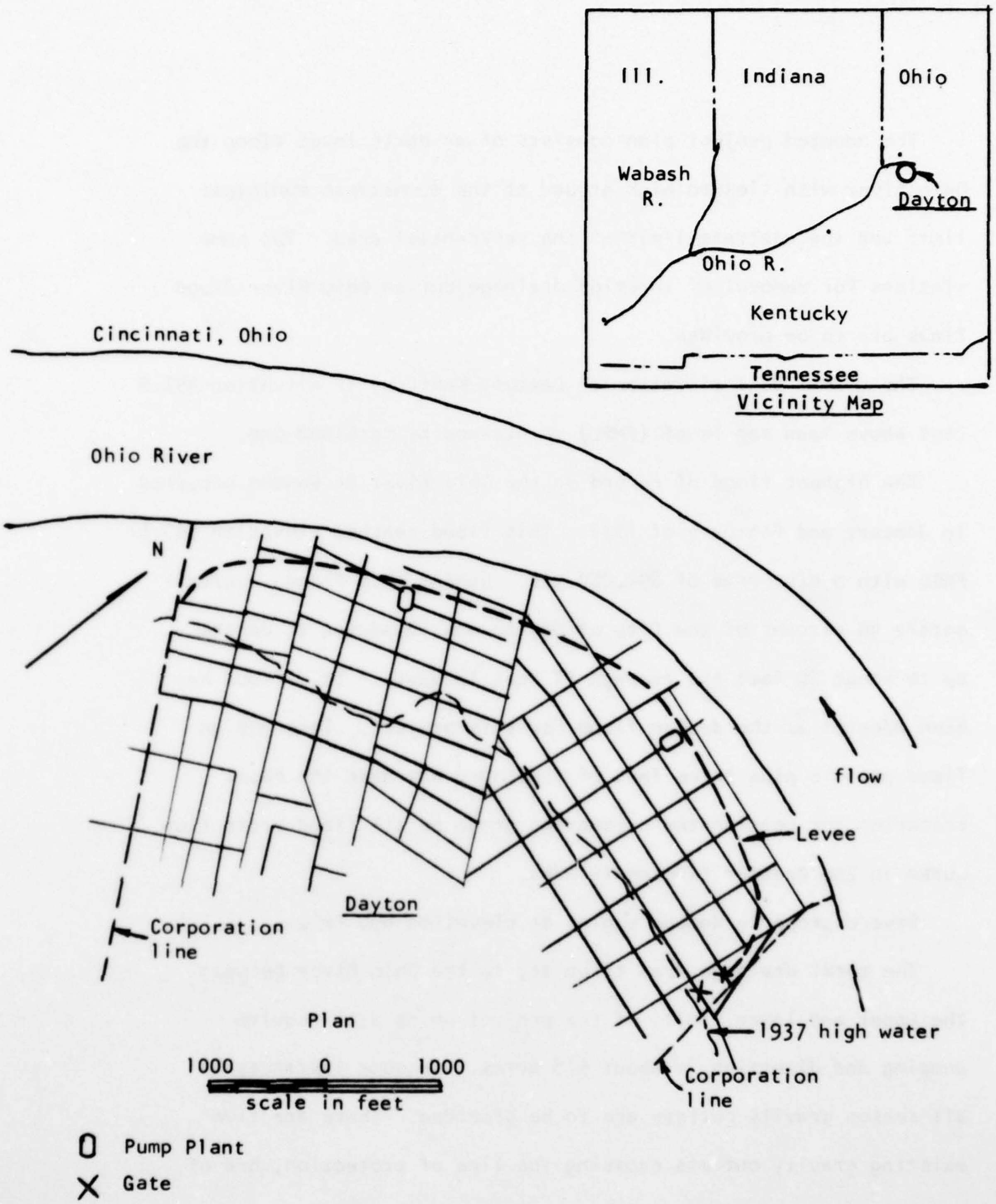


Fig. 3. Dayton, Kentucky, local flood protection project. Vicinity map and plan

The adopted project plan consists of an earth levee along the Ohio River with ties to high ground at the downstream municipal limit and the upstream limit of the residential area. Two pump stations for removal of interior drainage during Ohio River flood flows are to be provided.

The normal pool elevation at Dayton, Kentucky is elevation 455.0 feet above mean sea level (FMSL) maintained by Markland Dam.

The highest flood of record on the Ohio River at Dayton occurred in January and February of 1937. This flood reached elevation 509.61 FMSL with a discharge of 894,000 cfs. During this flood, approximately 40 percent of the City of Dayton was inundated to depths up to about 20 feet and average 12 feet in depth. This flood has been adopted as the design flood for this project. This design flood profile plus three feet of freeboard has been the basic criterion for setting the protection grade on all flood protection works in the Greater Cincinnati Area.

Severe property damage begins at elevation 490 FMSL.

The total drainage area tributary to the Ohio River between the upper and lower limits of the project which will require pumping and diversion is about 518 acres. In some instances all-season gravity outlets are to be provided. There are five existing gravity outlets crossing the line of protection, one of which is for storm water while the others are combined sewers.

McKinney Street Pumping Station

The McKinney Street pumping station is to receive flood period runoff from 313 acres of heavily residential and commercial areas. The area is designated as Class II requiring 50 years degree of protection from interior runoff.

Main Street Pumping Station

This pumping station would receive flood period runoff from 205 acres of heavily residential and commercial area designated as Class II.

Supplementary sewerage systems are to be constructed which will convey interior drainage to the pumping plants. The interior drainage will pass through the levee by gravity when Ohio River flows permit. The quantity of interior drainage expected to pass by gravity was computed by use of the rational formula except where ponding was available. The intensity of a 50 year rainfall for a duration equal to the times of concentration in the various branches of the sewerage systems and runoff factors ranging in value from 0.3 to 0.6 were used in the formula.

When the Ohio River reaches elevation 478.0 FMSL at the Dayton site gravity drainage will no longer be possible. Pumps will be required to discharge interior drainage. The pumping capacity required is based on an application of the rational method using a rainfall whose frequency coincides with that determined by use of Chart D-2, EM 1110-2-1410. The runoff factor was assumed to be 0.7 during flood periods. Based on these values and the

drainage areas involved, a pumping plant discharge of 416 cfs was provided for at the McKinney Street pumping plant and a design discharge of 285 cfs was provided for in the design of the Main Street pumping plant.

Runoff coefficients proposed by Da Costa are plotted against percent impervious area in Figure 4. By determining the percent impervious area of the Dayton drainage area the Da Costa runoff coefficient may be quickly determined. The runoff coefficient is in each case somewhat less in value than the value for the percent of impervious area.

It is not known to what extent future urbanization will affect the degree of protection provided by this project. The runoff coefficients used appear conservative enough to allow some degree of urbanization before the degree of protection is significantly affected. However, additional quantitative information pertaining to the effects of urbanization on the rational method would permit more accurate design of drainage facilities.

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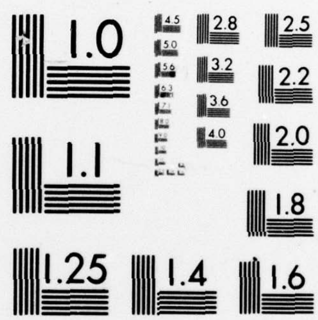

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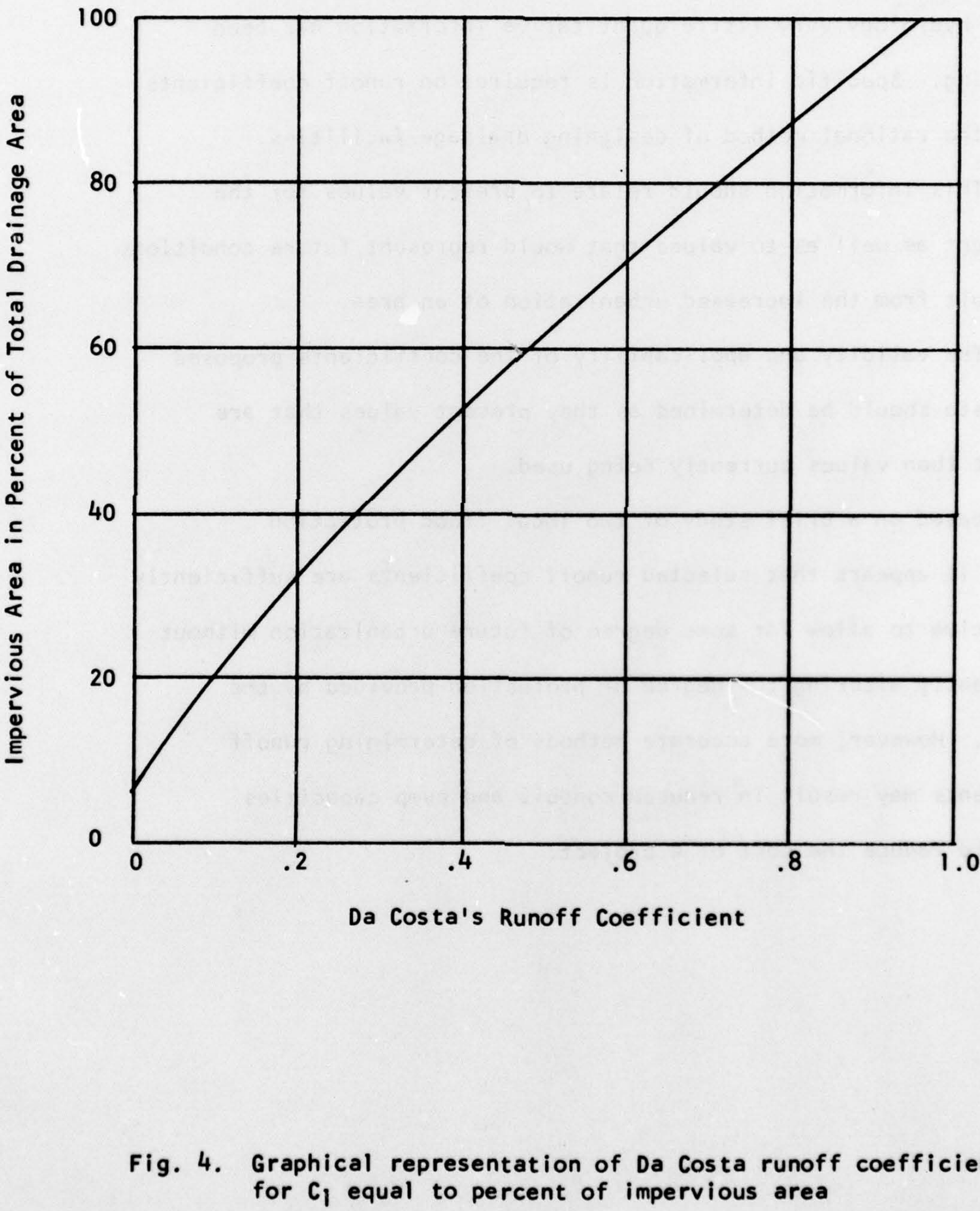

Fig. 4. Graphical representation of Da Costa runoff coefficients for C_1 equal to percent of impervious area

IV. Conclusions.

1. Although there is much research underway on the subject of urban hydrology very little quantitative information has been forthcoming. Specific information is required on runoff coefficients used in the rational method of designing drainage facilities.

2. This information should relate to present values for the coefficient as well as to values that would represent future conditions that result from the increased urbanization of an area.

3. The validity and applicability of the coefficients proposed by Da Costa should be determined as they present values that are different than values currently being used.

4. Based on a brief study of two local flood protection projects it appears that selected runoff coefficients are sufficiently conservative to allow for some degree of future urbanization without significantly altering the degree of protection provided by the projects. However, more accurate methods of determining runoff coefficients may result in reduced conduit and pump capacities that could reduce the cost of a project.

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URBAN HYDROLOGY CONSIDERATIONS IN THE
DESIGN OF INTERIOR DRAINAGE FACILITIES
FOR LOCAL FLOOD PROTECTION PROJECTS

Discussion

Question, Mr. Beard: If a pump-station size is reduced in the design stage, savings in reduced cost will be offset to some extent by increased frequency of damage in the protected area. How were the degrees of protection shown in Table 4 selected?

Reply, Mr. Holler: Degrees of protection shown were taken directly from EM 1110-2-1410 and result from research conducted by OCE.

Comment, Mr. Matthias: In addition to "C", another item of uncertainty in the Rational Formula is the "I" which is dependent on the time of concentration. The time of concentration is also not fixed and subject to different estimates by different persons.

Reply, Mr. Holler: Agree. The only sure value in the use of the Rational Equation appears to be the value used for "A". Information is required to more accurately determine the time of concentration.

Comment, Mr. K. Johnson: It may be useful to elaborate on the computation of peak discharges using the critical combination of intensity and drainage area contributing for selected times of concentration shorter than the total time for the entire watershed.

Reply, Mr. Holler: In the case of the Dayton LFPP this was considered. For Middlesboro it was not.

SUMMARY AND CONCLUSIONS

by

Leo R. Beard

Papers presented at the seminar and discussions were primarily related to the quantitative effects of urbanization on surface runoff. The effects on runoff quality and on ground water have apparently not been of appreciable concern in the Corps of Engineers studies represented at this seminar. No discussion of backwater studies and very little discussion of loss functions for pervious areas has been included.

From the presentations and discussions, it is apparent that urban runoff problems are becoming dominant in many Corps of Engineers studies. Whereas they have been incidental in past studies, particularly in connection with interior drainage problems, they are now considered as problems in themselves in many cases. There is still considerable practice of adopting a particular degree of protection (such as 10-year flood for a sewer lateral) as an engineering standard, depending on the nature of urban development, but there are also some efforts being made to determine economic efficiencies of alternative degrees of protection in the design of drainage facilities.

The papers presented represent a great variety of problems in urban drainage, and the material represents a broad spectrum of experience in the management of urban runoff.

Although the unit hydrograph technique is used in most studies, the Rational Method of runoff computation is still successfully used in some cases, particularly for preliminary studies. An interesting version of this method was introduced by Mr. Holler, who described work done by Mr. Da Costa in Portugal. Practically no work has yet been done by the Corps using more elaborate models such as the Stanford Watershed model. Even in those cases where the unit hydrograph technique is applied, a variety of computation procedures is represented.

The effects of urbanization on surface runoff are generally treated as an increase in runoff volume due to sealing pervious areas and a decrease in time of runoff due to increased conveyance efficiency. In general, criteria applied are derived empirically. It is generally recognized that the urban effects differ with different storm intensities.

The relationships between unit hydrograph parameters and basin characteristics that have been discussed differ appreciably. Basin characteristics usually include a stream length, as used by Snyder, and usually stream slope or basin slope. The more elaborate relationships include several other basin characteristics.

A common complaint is that there is a general lack of data in the areas being studied. This requires that generalized studies by made and information transferred from one area to another. Usually even this is not feasible to a satisfactory degree, and considerable implementation is necessary. In this connection, it has been pointed out that there is immediate need to assemble available data on urban runoff and available criteria for the design of urban drainage systems. The need for a long-range development program was also pointed out, such a program to include the gathering of basic data, the development of better methods and the formulation of design criteria.

An immediate need exists for adopting a unified method for computing urban runoff that can be used by agencies having a common interest. At present, the work of the various coordinating agencies is usually inconsistent and the Corps particularly has a problem of designing major drainage structures whose capacity must correspond to various elements of the main sewer systems.

The question of rapidly draining areas at the upper end of the system or, alternatively, providing for some storage at or above the intakes, was discussed considerably. It appears that the problems created by rapid drainage are increasing beyond a tolerable limit in many regions and that there is an increasing need for a balanced overall system design. It might be logical to require that design flows in the system should not exceed flows of comparable frequency that existed under natural conditions, but it would probably be difficult to implement this in many cases. At a very minimum, it appears essential to balance the costs and damages due to upstream storage against the costs and damages due to downstream discharges.

A number of computer programs exist that can be used in the computation of urban runoff. Most of these use the unit hydrograph technique. The comprehensive program of The Hydrologic Engineering Center is particularly adaptable for the computation of flows throughout a drainage system under natural and urbanized conditions and for evaluating average annual damages at pertinent locations. It is felt that future compu.. ~ orograms will probably depart from the unit hydrograph technique, acc..pt distributed inputs (non-uniform rainfall distributions), account for sewer capacities and associated pondage and overflow, and include generalized criteria for computing runoff in ungaged areas.

Conclusions that appear to be appropriate as a result of this seminar are:

1. The Corps of Engineers should assemble and coordinate available design criteria for use by all offices.

2. Steps should be taken to coordinate the overall design of urban drainage improvements to assure a balance between storage and discharge within the system.

3. The Corps of Engineers should sponsor a long-range development program including the collection and processing of basic data on urban runoff (probably administered by the U.S. Geological Survey), the development of improved methods for computing urban runoff, and the formulation of generalized criteria that can be applied in the design of urban drainage systems throughout the United States.

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4. TITLE (*and Subtitle*) Proceedings of a Seminar on Urban Hydrology		5. TYPE OF REPORT & PERIOD COVERED
		6. PERFORMING ORG. REPORT NUMBER
7. AUTHOR(s) E.F. Childs, G.S. Hare, D.L. Robey, W.R. Henson, S.E. Jones, T.L. Nelson, C.W. Timberman, C.D. Mat-thias, J.A. Constant, K. Johnson, J.B. Clark, etc.		8. CONTRACT OR GRANT NUMBER(s)
9. PERFORMING ORGANIZATION NAME AND ADDRESS The Hydrologic Engineering Center US Army Corps of Engineers 609 Second Street, Suite I, Davis, CA 95616		10. PROGRAM ELEMENT, PROJECT, TASK AREA & WORK UNIT NUMBERS
11. CONTROLLING OFFICE NAME AND ADDRESS		12. REPORT DATE September 1970
		13. NUMBER OF PAGES 308
14. MONITORING AGENCY NAME & ADDRESS(*if different from Controlling Office*)		15. SECURITY CLASS. (*of this report*) Unclassified
		15a. DECLASSIFICATION/DOWNGRADING SCHEDULE

16. DISTRIBUTION STATEMENT (*of this Report*)

Distribution of this publication is unlimited.

17. DISTRIBUTION STATEMENT (*of the abstract entered in Block 20, if different from Report*)

18. SUPPLEMENTARY NOTES

19. KEY WORDS (*Continue on reverse side if necessary and identify by block number*)

Urbanization, flood control, unit hydrographs, storm runoff

20. ABSTRACT (*Continue on reverse side if necessary and identify by block number*)

The seminar proceedings contain thirteen technical papers which presented by Corps of Engineers on 1-3 September at The Hydrologic Engineering Center. The papers discuss current problems in urban hydrology, review methods and techniques being used, and focus on future research needs. Titles and authors of papers are as shown

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TITLE	AUTHOR
Effects of Urban Expansion on Hydrologic Investigations	Elliot F. Childs
Effects of Urban Development on Storm Runoff Rates	George S. Hare
Effects of Urbanization on Annual Peak Flow Frequency Analysis	Donald L. Robey
A Unified Method for Computing Peak Discharges from Ungaged Urban Areas	William R. Henson
Tulsa District Methods of Urban Hdyrology	Stenley E. Jones
Synthetic Unit Hydrograph Realtionships Trinity River Tributaries, Forth Worth - Dallas Urban Area	Thomas L. Nelson
Discussion of Some Aspects of Urban Hdyrology Methodology	C.W. Timberman
Urban Hydrology in Connection with Channel Improvement Project at Newmarket Creek, Virginia	Carl D. Matthias
A Mathematical Determination of the Ordinates of the Unit Hydrograph	James A. Constant
An Analysis of the Effects of Urbanization on Unit Hydrograph Characteristics, Antelope Creek, Lincoln, Nebraska	Keith Johnson
Urban Hydrology Considerations - State of Hawaii	James B. Clark
Kansas City District Experiences in Urban Hydrology	W.L. Northrop
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